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# PROCEEDINGS

## American Society

of

## Civil Engineers

SEPTEMBER

1932

VOLUME 58 NUMBER 7

# PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 58 SEPTEMBER, 1932 No. 7

CURRENT

## TECHNICAL PAPERS
## DISCUSSIONS
## APPLICATIONS FOR ADMISSION AND TRANSFER

Published monthly, except June and July, at 99–129 North Broadway, Albany, N. Y., by the American Society of Civil Engineers, Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, December 28, 1931, at the Post Office at Albany, N. Y., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

# TESTS OF RIVETED AND WELDED STEEL COLUMNS

BY WILLIS A. SLATER,[1] AND M. O. FULLER,[2]
MEMBERS, AM. SOC. C. E.

SYNOPSIS

The tests reported in this paper were undertaken to secure a comparison of the behavior and strength of built-up steel columns fabricated by riveting, with similar columns fabricated by welding. The program included nine columns, of which two were riveted and seven were welded. In four of the welded columns, the welding was continuous throughout the length. In the other three welded columns, intermittent (or stitch) welding was used. The lengths (to the nearest inch) were 19 ft. 8 in. for three of the columns, 16 ft. 6 in. for two, 15 ft. 5 in. for three, and 5 ft. 6 in. for one.

The columns were built-up by riveting or welding cover-plates to the **H**-section or the **I**-section which formed the core. The total cross-sectional area ranged from 14.26 to 24.3 sq. in. in different columns. All were tested as flat-ended columns. Observations included measurement of strains, slip of plates, deflection of column due to the applied load, and strains caused by plastic flow of the heated metal and by the stress set up when the heated metal cooled. In general, the strains were measured in various positions in cross-sections at the top, middle, and bottom of the column. These observations gave the basis for determining the bending moment distribution between the ends and the center of the column. Tension test coupons from various positions in the cross-section were used to determine the variation of yield-point stress throughout the section.

A special feature was the testing of the column having an excessive initial curvature for comparison of strength, bending moment, and deflection, with those of a similar straight column.

NOTE.—Discussion on this paper will be closed in **December, 1932**, *Proceedings*.

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[2] Associate Prof., Civ. Eng., Lehigh Univ., Fritz Eng. Laboratory, Bethlehem, Pa.

## 1.—The Columns

The program herein reported, included the testing, during 1928 and 1929, of nine built-up steel columns, each consisting of a core, formed by an **I**-beam or an **H**-section, and a cover-plate attached to each flange of the core. Fig. 1 shows the details of the columns, including the methods of attaching the cover-plates to the flanges. In Columns Nos. 1, 1*BB*, 3, and 4, reinforcing plates

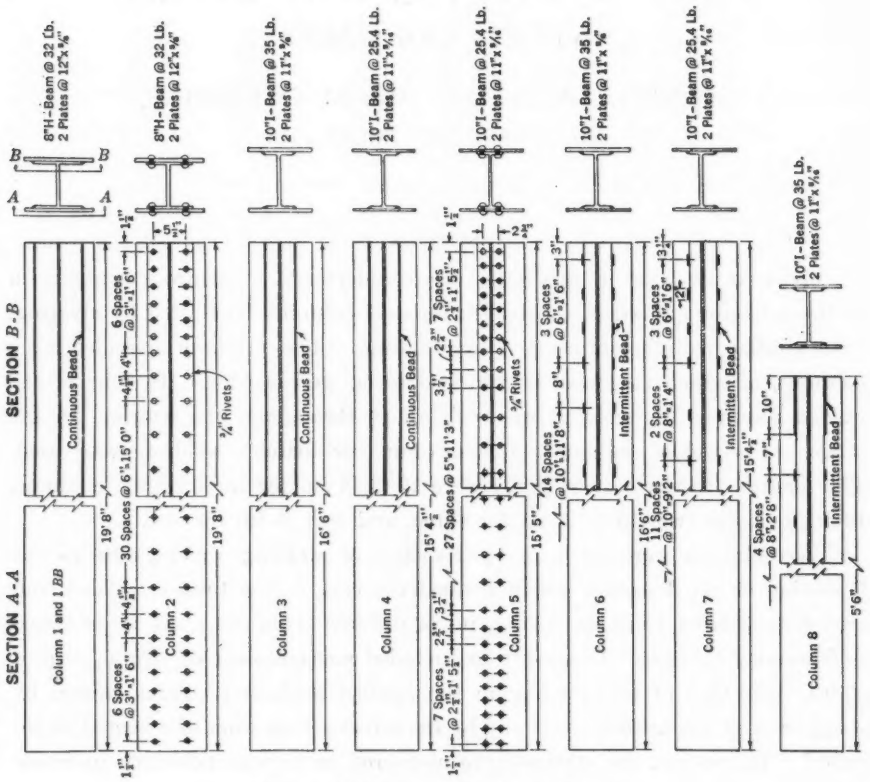


Fig. 1.—Details of Columns Tested.

were welded to the core section with a continuous head. In Columns Nos. 2 and 5, plates were riveted to the core section. In Columns Nos. 6, 7, and 8, plates were welded to the core section with an intermittent bead.

Column No. 1 was so much out of alignment that a new column, No. 1*BB*, was furnished to take its place. However, Column No. 1 was tested in order to study the effect of its lack of alignment on its behavior under load. Previous to testing each column, its departure from a straight line in the direction of each of two planes was measured, one plane perpendicular to, and the other parallel to, the plane of the web. These departures are shown in Fig. 2.

Except for differences in method of fabrication, Columns Nos. 4, 5, and 7, and Columns Nos. 3 and 6 were designed to be counterparts of columns of Types 6 and 6*A*, respectively, tested at the Bureau of Standards for the Special Committee of the American Society of Civil Engineers on Steel Columns and Struts.[3] The Special Committee provided three lengths in each

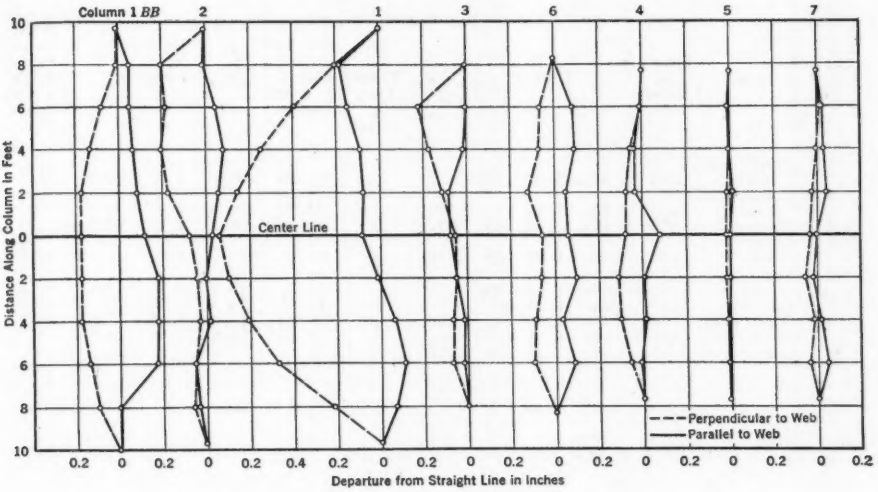


FIG. 2.—INITIAL ALIGNMENT OF COLUMNS.

type of column, and the columns here reported were originally of the intermediate length. However, in this series of tests, in order to provide material for coupons, 1 ft. was cut from the end of each column, making them 1 ft. shorter than the corresponding columns tested for the Special Committee.

The columns were tested with "flat ends" as nearly as this condition could be approached. The top and bottom of each column had been milled, but the milling did not give surfaces as nearly plane as might be desired. It is believed that lack of planeness of the ends affected somewhat, the location of the load line in the columns as the loads were applied. Details of methods of testing are given in the Appendix. The data of the test specimens are given in Table 1.

*Nomenclature.*—The term, "strain line," introduced in this paper, is used to designate the markings that appeared in the white cement coat (which had been applied to all the columns prior to testing) when the mill scale flaked off due apparently to the condition of stress in the steel. The expression, "unit deformation," and the word, "strain," are used to mean the change of length per unit of length, and the word, "stress," to mean the intensity of the internal force per unit of sectional area.[4] The expression, "unit deformation," is applied to a change of length due to any cause such as stress, or expansion due to heat, while the word, "strain," should be applied only to a change of

[3] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1598.
[4] *Proceedings*, Am. Soc. for Testing Materials, Vol. 25 (1925), Pt. I, p. 897.

length caused by stress. However, it is not always possible to determine what is the principal cause of a change of length and there may be some confusion in the use of the two terms. In the following notes the term "inside" or

TABLE 1.—DATA OF TEST COLUMNS

| Column No. | Type | Length, in inches | Core | Plates | | Sectional area of column, in square inches | $I$, in inches⁴ | $r$, in inches | $\frac{l}{r}$ | Average stress at maximum load, in kips* per square inch |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Width, in inches | Thickness, in inches | | | | | |
| 1 | Continuous weld.... | 236 | 8-in. H, 32 -lb. | 12 | $\frac{5}{8}$ | 24.30 | 206.6 | 2.92 | 80.8 | 32.3 |
| 1*BB* | Continuous weld.... | 236 | 8-in. H, 32 -lb. | 12 | $\frac{5}{8}$ | 24.30 | 206.6 | 2.92 | 80.8 | 32.3 |
| 2 | Riveted... | 236 | 8-in. H, 32 -lb. | 12 | $\frac{5}{8}$ | 24.30 | 206.6 | 2.92 | 80.8 | 32.5 |
| 3 | Continuous weld.... | 198 | 10-in. I, 35 -lb. | 11 | $\frac{5}{8}$ | 23.97 | 155.0 | 2.54 | 77.9 | 30.8 |
| 4 | Continuous weld.... | 184.5 | 10-in. I, 25.4-lb. | 11 | $\frac{5}{16}$ | 14.26 | 78.6 | 2.35 | 78.7 | 36.9 |
| 5 | Riveted... | 185 | 10-in. I, 25.4-lb. | 11 | $\frac{5}{16}$ | 14.26 | 78.6 | 2.35 | 78.7 | 35.8 |
| 6 | Stitch weld | 198 | 10-in. I, 35 -lb. | 11 | $\frac{5}{8}$ | 23.97 | 155.0 | 2.54 | 77.9 | 33.3 |
| 7 | Stitch weld | 184.5 | 10-in. I, 25.4-lb. | 11 | $\frac{5}{16}$ | 14.26 | 78.6 | 2.35 | 78.7 | 36.7 |
| 8 | Stitch weld | 65 | 10-in. I, 35 -lb. | 11 | $\frac{5}{16}$ | 17.10 | 77.8 | 2.12 | 31.1 | † |

* One kip equals 1 000 lb. †Not tested to failure.

"interior" face of a cover-plate or of a flange, refers to the face toward the center of the column. Similarly, "outside" or "exterior" face refers to the face away from the center of the column.

## 2.—PHENOMENA OF TESTS

The visual phenomena of the tests consisted principally of the formation of the "strain lines" in the whitewash caused by the buckling or flaking of the mill scale. Their occurrence is described in considerable detail for Column No. 4 which may be considered as representative. The significance of the strain lines in general is discussed in Section 4.

Strain lines first appeared in Column No. 4 (welded) at an applied load of 266 kips (approximately 19 kips per sq. in.). They appeared along lines extending horizontally from the edges toward the centers of the cover-plates for a distance of about 2 in. None was appreciably longer than 2 in. Strain lines were first observed near the top after which they extended rapidly over the entire length of the column. They occurred in pairs, one appearing on the outside face of the cover-plate for each one on the inside face. The vertical distances between the strain lines on the outside and inside faces were seldom more than $\frac{1}{8}$ in. With the machine stopped at a load of 266 kips, the number of strain lines continued to increase, but did not extend beyond 2 in. from the edge of the plate, nor did any appear on the I-section. A few forked slightly near their ends.

Under an applied load of 308 kips (approximately 22 kips per sq. in.) strain lines appeared in the I-section, extending from the fillets outward toward the edges of the flanges (Fig. 3). These strain lines at first were about $\frac{3}{8}$ in. long. As the load increased they extended slightly, but never reached

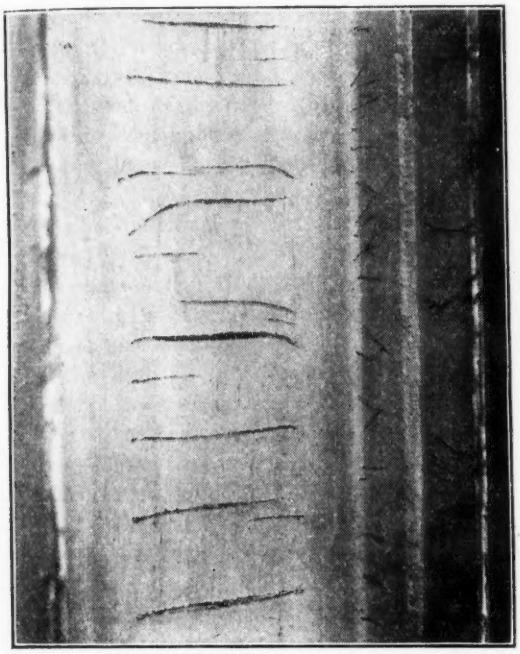

FIG. 3.—STRAIN LINES ON WEB OF COLUMN NO. 4.

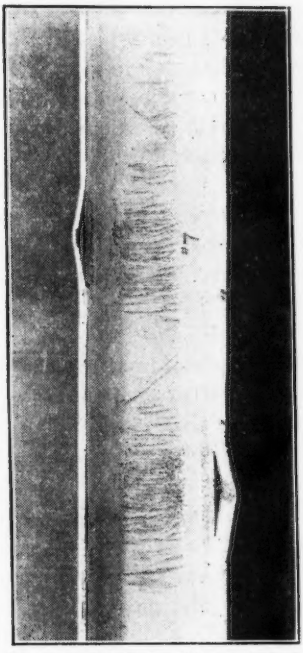

FIG. 4.—COLUMN NO. 7 AFTER TEST SHOWING STRAIN LINES AND BUILDING OF COVER-PLATES.

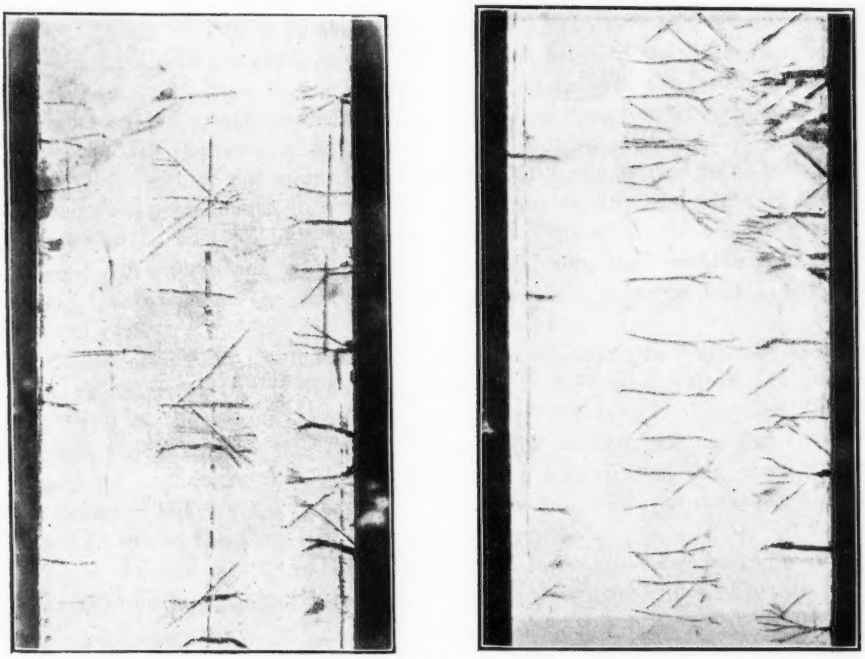

FIG. 5.—STRAIN LINES ON EXTERIOR FACES OF COVER-PLATES OF COLUMN NO. 4 AFTER TEST.

a length greater than $1\frac{1}{4}$ in., and did not extend into the web. Some of them were inclined, but generally they were horizontal. At the same load (308 kips) strain lines were found on the outside of the cover-plates crossing the plane of the web at the top within about 3 in. of the end of the column. As the loading continued, these strain lines extended and became more pronounced. Those that had developed earlier in the test showed more forking at the ends.

The distribution of strain lines after the completion of the test is well shown in Figs. 3 and 5. The flanking of the whitewash merely accentuated the phenomenon of the separation of the mill scale from the steel. At places, patches of mill scale of considerable size separated from the metal. In Fig. 3 the shapes of the strain lines are shown for the web and the inner faces of the flange. The uniformity with which the strain lines in the web of the columns extended, about 2.5 in. on either side of the vertical center line of the column, is notable, as well as the regularity with which the strain lines in the flanges began at the fillet and terminated at a vertical line about 1.5 in. from the fillet.

The location of strain lines on the exterior face of the cover-plates, corresponds closely with that of the strain lines on the interior face of the cover-plate and the flange of the I-beam. Most of the strain lines were horizontal at the time of reaching the maximum load. After reaching the maximum load the testing machine was kept running until the load had decreased to about 500 kips. The photographs were taken after the completion of the test, and figures formed by the flaking show many more diagonal lines and much more forking than were present at the maximum load. The vertical line that appears in Fig. 5 at about the position of the web of the column is not due to strain in the plate, but to accidental abrasion of the whitewash. There were no strain lines at the junction of the plate with the I-section. At the higher loads the column deflected markedly in a direction at right angles with the web. No other marked phenomenon of failure appeared.

With most of the columns, the cause of failure appeared to be a deflection over the entire length in a plane at right angles with the plane of the web. In Columns Nos. 5 and 7, however, local buckling of the cover-plates between points of attachment took place at the maximum load, and this apparently was the immediate cause of failure. Both these columns had 11 by $\frac{5}{16}$-in. cover-plates.

In Column No. 5 the buckling occurred on only one side, at about mid-height where the rivets were 5 in. apart in the vertical direction. It had been noticed at the beginning of the test, before any load was applied, that the plate which buckled was slightly bent at the bottom bearing, but there was nothing to indicate that this slight crippling had affected the behavior of the column. The average stress on the entire section was approximately 35.8 kips per sq. in. at the time that the buckling occurred.

In Column No. 7 the buckling occurred in both cover-plates, one at 10 in. and the other at 31 in. below mid-height, as shown in Fig. 4. In this column

the distance from center to center between the 2-in. stitch-welds, was 10 in. No defects in the welds at the points of buckling were observed. Buckling of the plates occurred in both the riveted and the stitch-welded columns with $\frac{5}{16}$-in. cover-plates, but did not occur in either the riveted or the stitch-welded column with $\frac{5}{8}$-in. cover-plates. The ratios of the rivet or weld spacing to the thickness of plates are given in Table 2.

TABLE 2.—RATIOS OF RIVET OR WELD SPACING TO THICKNESS OF PLATES

| Column No. | Type | Plate thickness, $t$, in inches | Center to center spacing, $s$, in inches | $\frac{s}{t}$ | Remarks |
|---|---|---|---|---|---|
| 2 | Riveted | $\frac{5}{8}$ | 6 | 9.6 | Plates did not buckle |
| 5 | Riveted | $\frac{5}{16}$ | 5 | 16 | One plate buckled |
| 6 | Stitch-welded | $\frac{5}{8}$ | 10 | 16 | Plates did not buckle |
| 7 | Stitch-welded | $\frac{5}{16}$ | 10 | 32 | Both plates buckled |

## 3.—SLIPPING OF PLATES

Gauge Lines *B*5, *B*11, *B*17, and *B*23 (Fig. 6), were placed so that their changes in length indicated the movement of a point on the cover-plate relative to a point on the **I**-section or **H**-section. These gauge lines were placed only on Section *B* at the top of Column No. 4, and on both Sections *B* and *D*

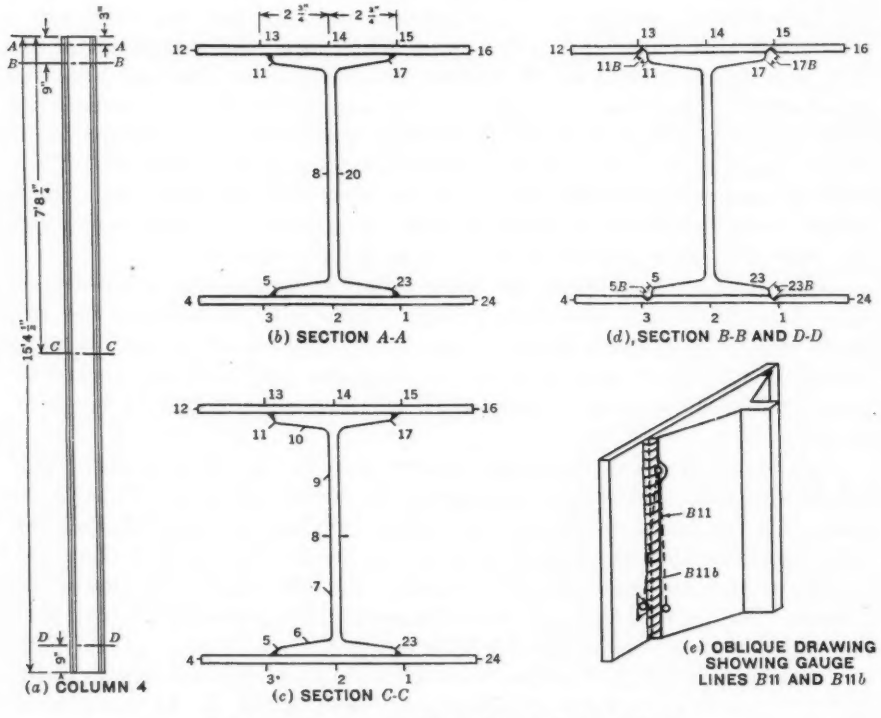


FIG. 6.—LOCATION OF GAUGE LINES, COLUMN NO. 4.

for Columns Nos. 1*BB*, 2, 3, 5, 6, and 7, since it was believed that if slipping occurred at all, it would be near one end of the column. Gauge Lines *B*11 and *B*11*b* are shown in Fig. 6(*e*). Gauge Lines *B*5, *B*5*b*, *B*17, *B*17*b*, *B*23, and *B*23*b*, are similarly arranged. Fig. 7 presents some representative curves showing the movement in various gauge lines. They have been plotted in pairs, both gauge lines of each pair being referred to the same origin. The

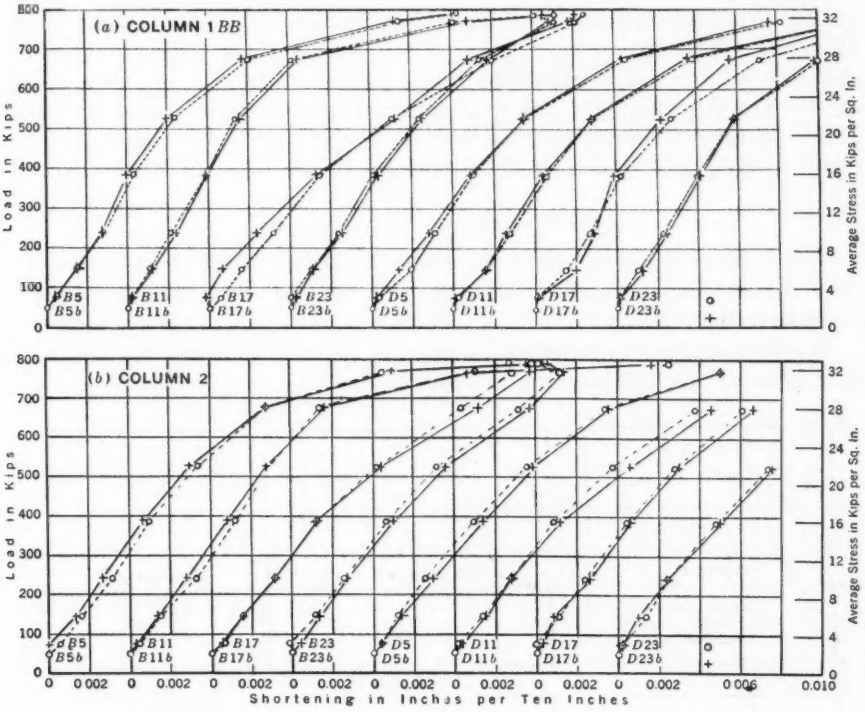


FIG. 7.—STRESS-STRAIN CURVES FOR DETERMINATION OF SLIP OF PLATES.

dotted lines are the graphs for Gauge Lines 5, 11, 17, and 23, and the solid lines are for Gauge Lines 5*b*, 11*b*, 17*b*, and 23*b*. The numbers designating the gauge lines are shown near the origin of the respective graphs. The capital letters, *B* or *D*, indicate whether the measurement was at Section *B* or Section *D*, and the Arabic numerals indicate the position of the gauge line in the section. (See Fig. 6.) Similar sets of curves were drawn for the other columns.

If there were no slipping of the plates, and no error of observation, the dotted and the solid lines should coincide. Any divergence indicates either error of observation or slipping of the plates, or both. Since there is certain to be some error of observation, some divergence of the lines is inevitable. If an error occurs in the zero reading, it will cause the dotted and the solid graphs to remain at a fixed horizontal distance apart; if it occurs in readings for loads above that for the zero reading, it will cause an erratic divergence

and convergence of the graphs. Slipping of the plate with reference to the structural shape should be expected to increase with increased load, and, therefore, should cause a progressive divergence of the graphs.

The curves, including those in Fig. 7, show many erratic divergences and convergences of the lines, most of which are slight, indicating slight errors in the measurements. There are a few slight divergences which are progressive over a part of the test, such as that for Gauge Lines *D*17 and *D*17*b*, for Column No. 1*BB*, and that for Gauge Lines *D*11 and *D*11*b*, for Column No. 2 (Fig. 7). These divergences, however, represent at most a slip of approximately 0.001 in., and it cannot be stated positively that it represents a slip at all. On the whole, the practical coincidence of the dotted and the solid graphs in these diagrams indicate that there was little if any slipping of the plates in either the welded or the riveted columns.

### 4.—Strains and Strain Lines

Fig. 6 shows the positions of the gauge lines. While the cross-sections of the different columns varied, the same plan of gauge numbering was used throughout the series.

Bending strains developed quite early in the test, although the bearings were adjusted in such a way that, at an applied load of approximately 60 kips,

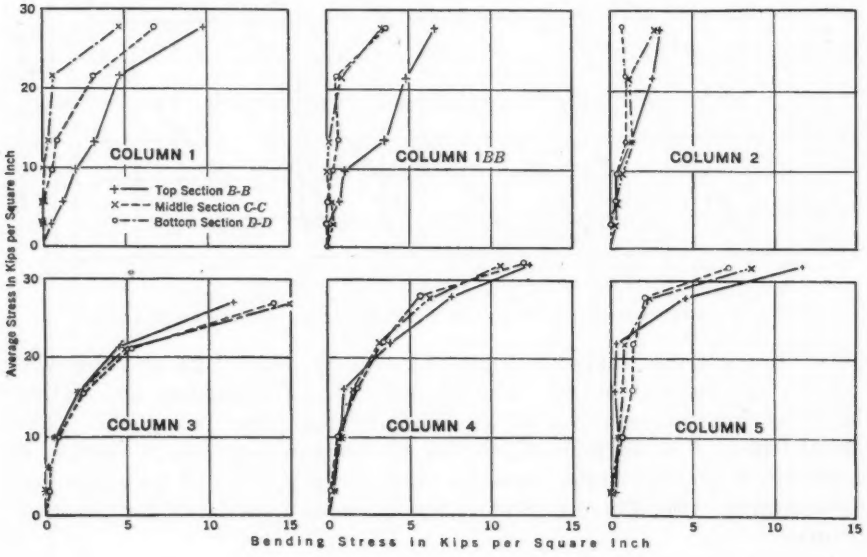


Fig. 8.—Bending Stresses in Columns.

the observed strains at the four corners of the column were equal. It was assumed that the over-all shortening, which was used in making the adjustments of the bearing (see Appendix, Article 2), would detect bending strains in the columns due to eccentricity of the bearing. However, an examination of Column No. 4 (see Fig. 8), indicated bending moments at the middle and

at the ends nearly equal in value and opposite in sign. This reversal of moment tended to equalize the over-all shortening measured on the four corners of the column, and, therefore, reduced the effectiveness of these mea-

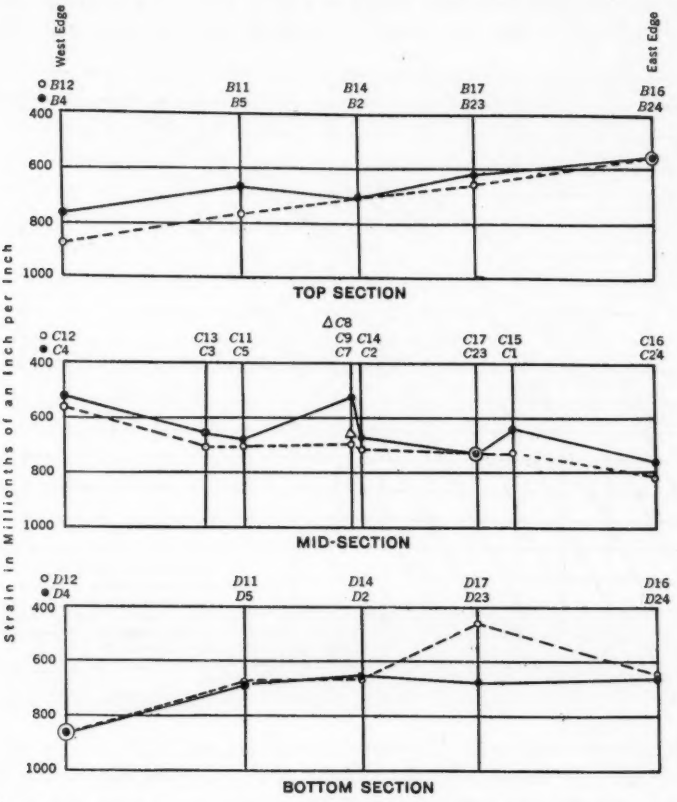


FIG. 9.—DISTRIBUTION OF STRAINS AT SECTIONS OF COLUMN NO. 4 FOR A LOAD OF 308 KIPS, OR A STRESS OF 216 KIPS PER SQUARE INCH.

surements as an adjusting agent. It is evident that if the over-all shortening at the four corners of the column were equal in spite of large bending moments in the column, the measurement of this shortening would be of no use in detecting eccentricity of load.

The presence of bending strains is indicated by the slope of the graphs in Fig. 9, which shows the total measured strains for the cross-sections at *BB*, *CC*, and *DD* of Column No. 4. Fig. 10 shows a pronounced change of slope of the average stress-strain curve at the stress of 21.6 kips per sq. in. Fig. 9 is more or less typical of the strain distribution across the sections of the columns for loads near the critical.

Up to a certain load for each column, the average strain at mid-section was approximately the same as the average for the full length of the column. Above this load the average strain at the center was greater than the average

for the full length of the column. This is shown by the divergence of the broken from the solid lines, for the higher loads in Fig. 10.

Figs. 3, 4, 5, 11, 12, 13, 14, and 15 are representative views of the strain lines that appeared in columns of various types and designs during the testing. They were all taken after the completion of the individual tests. Evidently,

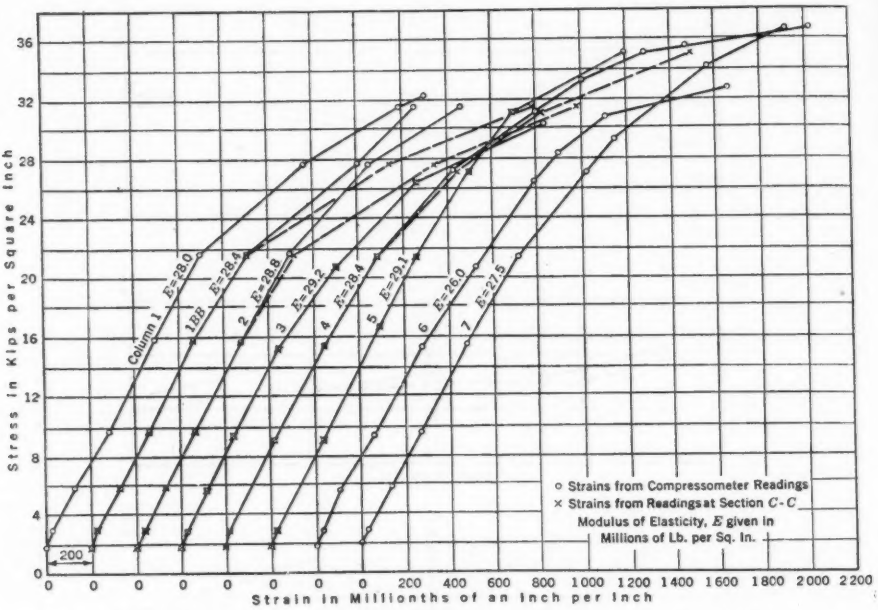
Stress in Kips per Square Inch
Strain in Millionths of an Inch per Inch
Column 1 E=28.0
1BB E=28.4
2 E=28.8
3 E=29.2
4 E=28.4
5 E=29.1
6 E=26.0
7 E=27.5
200
o Strains from Compressometer Readings
× Strains from Readings at Section C-C
Modulus of Elasticity, E given in Millions of Lb. per Sq. In.

FIG. 10.—AVERAGE STRESS-STRAIN CURVES FOR COLUMNS NOS. 1 TO 7.

two major influences affected the number, location, and direction of the strain lines in the whitewash, namely, the weight of the section, and the method of fabrication. In the column of the lighter sections, the strain lines were much more general than with the heavier sections. With the 10-in., 25.4-lb. I-sections, the strain lines in the web were closely spaced and prominent, but never extended into the fillets, either from the webs or from the flanges. (See Figs. 3 and 4.) With both the heavier cores; that is, the 8-in., 32-lb. H-sections (Fig. 14) and the 10-in., 35-lb. I-sections, the strain lines were less general, but they extended entirely across the web and out into the flanges.

No case was found in which a strain line crossed a weld, and only a few in which they crossed the line of the weld on the outer face of the plate; that is, on the face opposite that of the weld (Fig. 15). In the stitch-welded columns (Fig. 4), the same general placement of the strain lines existed as in the columns with the continuous welds, except that in the former the strain lines, generally, were found in groups opposite the welds and seldom in the sections between the intermittent welds. With the riveted columns, the strain lines extended to the edge of the flange and up close to the rivets. (See Figs. 11 and 12.)

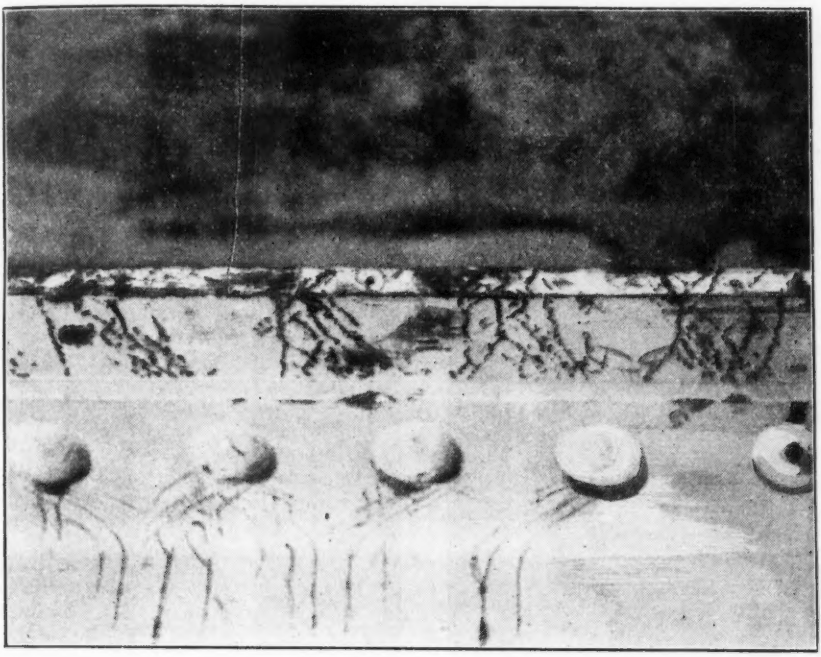

FIG. 12.—STRAIN LINES ON INTERIOR FACE OF COVER-PLATE OF COLUMN NO. 2 AFTER TEST.

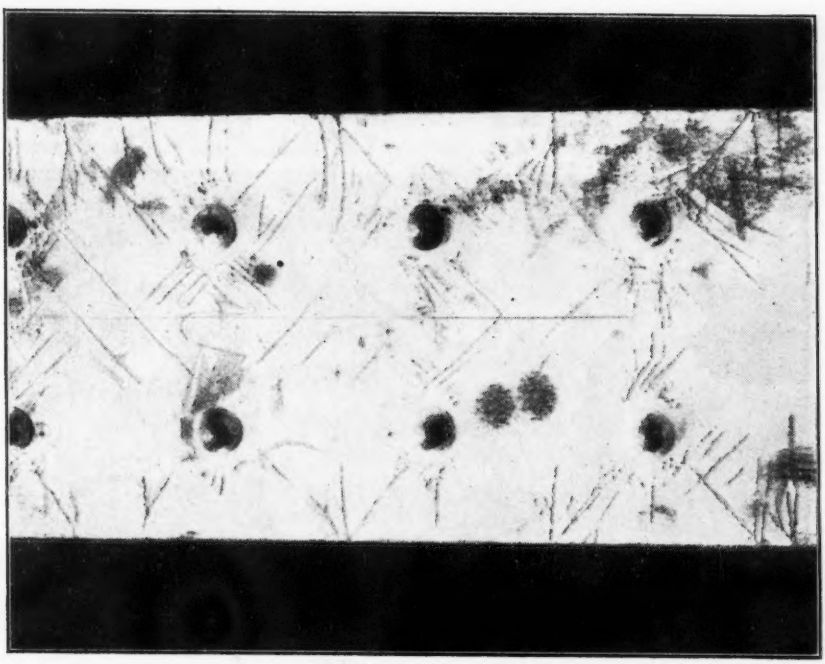

FIG. 11.—STRAIN LINES ON EXTERIOR FACE OF COVER-PLATE OF COLUMN NO. 2 AFTER TEST.

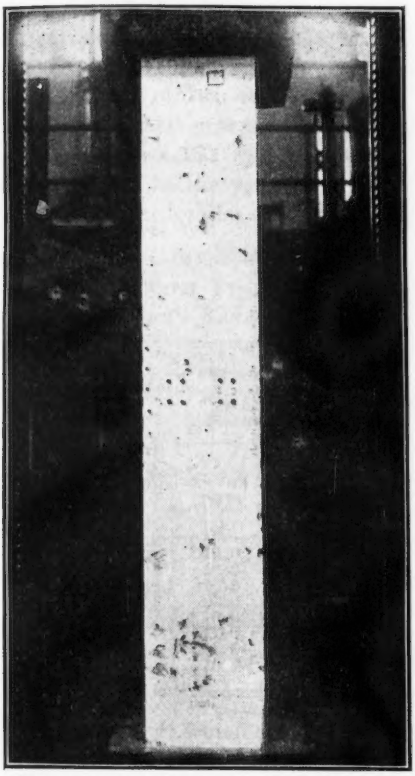

FIG. 13.—STRAIN LINES ON EXTERIOR FACE OF COVER-PLATE OF COLUMN NO. 8 AFTER LOADING TO AVERAGE STRESS OF 27.7 KIPS PER SQ. IN.

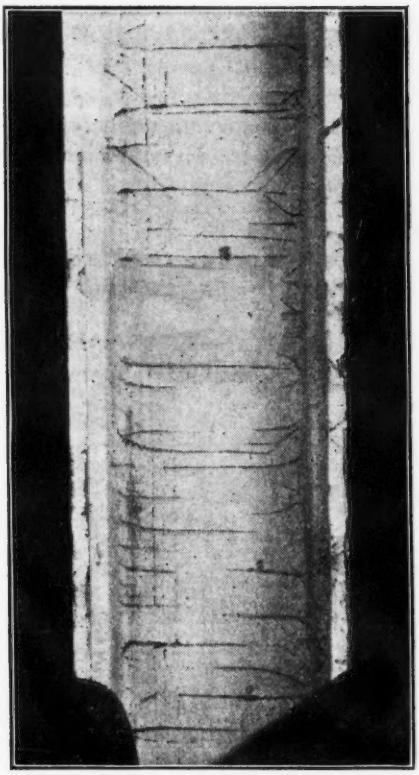

FIG. 14.—STRAIN LINES ON WEB OF COLUMN NO. 1 *BB* AFTER TEST.

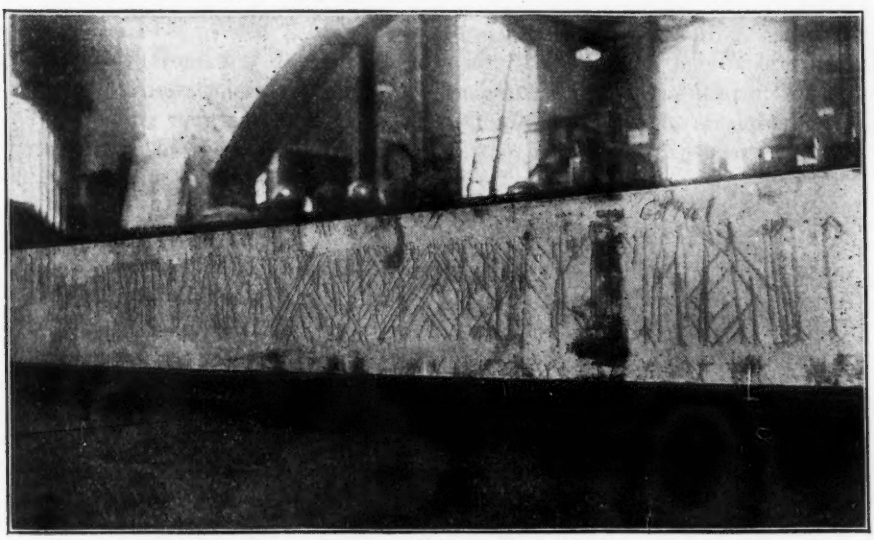

FIG. 15.—STRAIN LINES ON EXTERIOR FACE OF COVER-PLATE OF COLUMN NO. 1 AT MAXIMUM LOAD.

In each of the columns the first strain line appeared in one of the cover-plates, generally near the edge, at the stress given in Table 3, Column No. 4. The stress at this time ranged from 21 to 10 kips per sq. in., except for Column No. 6, in which the first strain lines appeared at a stress of 5.8 kips per sq. in. The average stress at which a strain line first appeared in the cover-plates was 15.1 kips per sq. in. The average stress when a strain line first appeared in the web is shown in Table 3 to have ranged from 36 to 22 kips per sq. in. for the various columns.

TABLE 3.—COMPARISON OF COMPUTED STRESSES FOR MAXIMUM LOAD ON COLUMN WITH YIELD POINT STRESSES FROM COUPONS AND WITH STRESSES COMPUTED BY RANKINE'S FORMULA

| Column No. | Yield point of coupons, in kips per square inch | Stress at First Strain Line, in Kips per Square Inch | | Stress at proportional limit of column, in kips per square inch | Stress at Maximum Load, in Kips per Square Inch | | Ratio of stress at maximum load to yield point of coupons | Ratio of stress from Rankine formula to stress at maximum load |
|---|---|---|---|---|---|---|---|---|
| | | Cover-plate | Web | | From test | Computed by Rankine formula | | |
| 1 | 34.9 | 16.0 | 22.0 | 21.8 | 32.3 | 27.7 | 0.925 | 0.858 |
| 1*BB* | 35.4 | 16.0 | 28.0 | 21.8 | 32.3 | 28.1 | 0.912 | 0.870 |
| 2 | 35.9 | 16.0 | 32.0 | 21.8 | 32.5 | 28.5 | 0.905 | 0.877 |
| 3 | 34.6 | 21.0 | 27.0 | 15.3 | 30.8 | 27.8 | 0.892 | 0.903 |
| 4 | 38.7 | 19.0 | 29.0 | 36.9 | 36.9 | 31.0 | 0.954 | 0.840 |
| 5 | 39.2 | 17.0 | 36.0 | 21.4 | 35.8 | 31.4 | 0.913 | 0.877 |
| 6 | 37.9 | 5.8 | 24.0 | 15.4 | 33.3 | 30.5 | 0.879 | 0.916 |
| 7 | 39.8 | 10.0 | 27.0 | 15.4 | 36.7 | 31.9 | 0.922 | 0.869 |

The stress at which strain lines appear is generally assumed to be approximately the yield point of the steel. For this to have been the case with these columns, requires that there must have been an initial stress at the point where the first strain lines started, averaging about 20 kips per sq. in. There are some reasons for thinking that compressive stresses were set up in the webs of the **I**-beams and **H**-beams by unequal cooling after rolling, but it is not possible from the data of these tests, to estimate their magnitude. Although initial stresses of considerable magnitude due to the heat of rolling and of welding were undoubtedly present, this fact need not be alarming since the strengths of the columns were about what should be expected with the materials, shapes, and lengths used.

### 5.—EFFECT OF WELDING ON STRAINS AND STRESSES

In a study of the strains due to welding, and their effect on the location of strain lines, Column No. 8 (5 ft. 6 in. long), was fabricated and tested. (See Fig. 13.) Two, 11 by $\frac{5}{16}$-in. cover-plates were stitch-welded to a core that consisted of a 10-in. 35-lb. **I**-beam. The welds at the top and bottom of the column were continuous for 10 in. from the ends. The intermediate welds were each 2 in. long and spaced 8 in. apart, center to center. Previous to the welding of the plates to the **I**-section, numerous 2-in. gauge lines were established and read. These lines were read again after the welding was completed and after the column had cooled down to normal temperature. Load was then

applied up to a maximum of 398 kips, which caused an average stress in the column of 27.7 kips per sq. in. After the strain readings had been obtained this load was removed and another set of readings was taken to measure any permanent set in the column.

Fig 16 shows the location of intermittent welds in relation to the gauge lines used to determine the distribution of the strains in a cross-section at the center of the column and along the edges of the cover-plate for some dis-

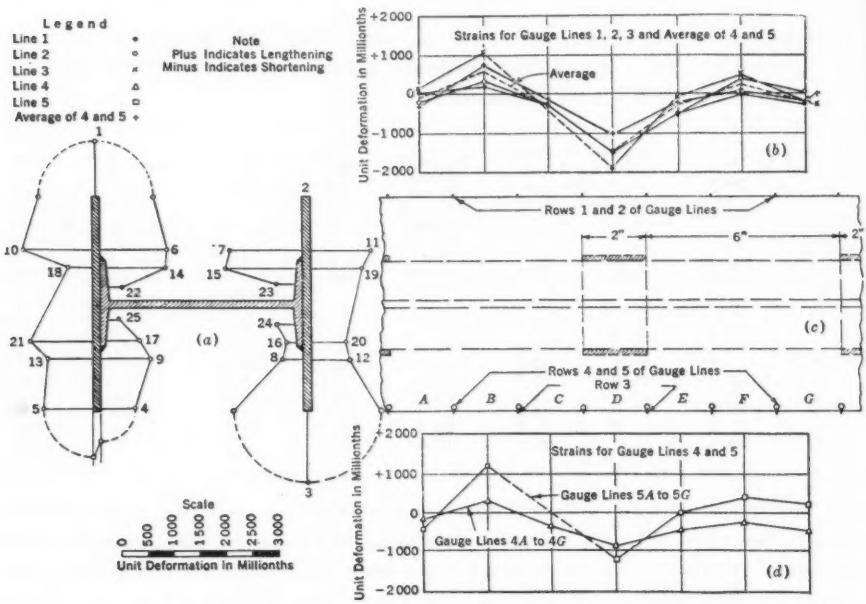


FIG. 16.—STRAINS IN COLUMN NO. 8 DUE TO WELDING.

tance above and below the center. The elevation of the gauge lines is given by letters, and the position on the cross-section, by numbers. For example, *D*-25 was at the mid-height of the column and on the fillet of the I-section.

In Fig. 16 (*a*), the unit deformations in all gauge lines cut by Section *D* at mid-height of the column have been plotted opposite the gauge line in a direction perpendicular to the surface in which the strain occurred. A distance away from the surface represents unit shortening. Evidently, all the gauge lines cut by the section were shortened, although it is unlikely that the stress was compression in all of them, since the total stress on the section was zero. There were no gauge lines on the web, but it does not seem likely that the web was in tension due to bending stresses.

Without some knowledge of how much plastic flow there was, it is impossible to state how many of the resultant deformations shown in Fig. 16 (*a*) represent stress. However, at points in the section so far removed from the weld that plastic flow due to the heat was unlikely, the strains indicate a compressive stress of about 15 kips per sq. in. and it seems likely that at the

weld the tensile stress was as great as the yield point of the metal, in spite of the fact that the strain readings showed that it had shortened due to the welding.

Fig. 16 (*b*) shows that at all four corners of the column there was a marked shortening at the section through the welds and a marked elongation at the sections between welds. Sections *B* and *F*, midway between welds, were far enough from the weld so that probably no plastic flow occurred in them. The average tensile strains at the edges of the plates for these two sections were 600 and 250 millionths, respectively. Using the average modulus of elasticity of 29 000 000 lb. per sq. in. the corresponding stresses are 17.4 and 7.25 kips per sq. in. Since the external force on each of these sections is zero, there must have been compressive stresses on other parts of the section to balance the tensile stresses.

The variations of strain along the edges of the plate, as shown in Fig. 16 (*b*), make evident why the strain lines on the welded columns appeared first opposite the welds. Evidently, the heat of welding had set up compressive stresses at these points and tensile stresses at the edges of the cover-plates between them, so that when the load came on the column, the yield point was reached first at points opposite the weld.

Fig. 16 (*d*) shows that the strains on opposite faces of the cover-plate were not equal, but that local bending strains, varying somewhat regularly from weld to weld, took place. These bending strains were probably due to the fact that the heat was applied to only one side of the plate. The difference in strains corresponds to a difference of stress on opposite sides of the plate, amounting at Section *B* to approximately 27, and, at Section *F*, to 19.5 kips per sq. in.

The phenomena pointed out in the foregoing paragraphs make it apparent that there is room for much further study of the effect of welding on stresses in structural members.

Upon application of load to the column, the strains were read and have been plotted in Fig. 17, in which the letters, *B*, *D*, and *F*, refer to positions shown in Fig. 16. The slope of the graphs indicates that the strains increased at approximately the rate to be expected for steel under stress, but beginning in each case with the strain set up by the welding. As a result, the total strains at Gauge Lines *D*1 to *D*21, under the final stress of 27.7 kips per sq. in., were very great. Strain lines formed in the white cement coat at the indicated points of high strain opposite to, and at some distance from, the welds, but none close to the welds. This is consistent with the fact that, as was previously pointed out, tensile stresses must have existed in the immediate vicinity of the welds due to shrinkage on cooling. The absence of strain lines in the welds may have a further explanation since auxiliary tests made on small angles showed no strain lines on the welds, although the load applied caused stresses far beyond the yield point of the steel.

### 6.—Modulus of Elasticity

Table 4 gives the moduli of elasticity as determined for certain coupons and for the columns. The moduli of elasticity of the coupons for the core

sections and the plates were weighted in proportion to the sectional areas of each, and the weighted average for the entire cross-section of the column was computed. To determine the moduli directly from the tests of the columns

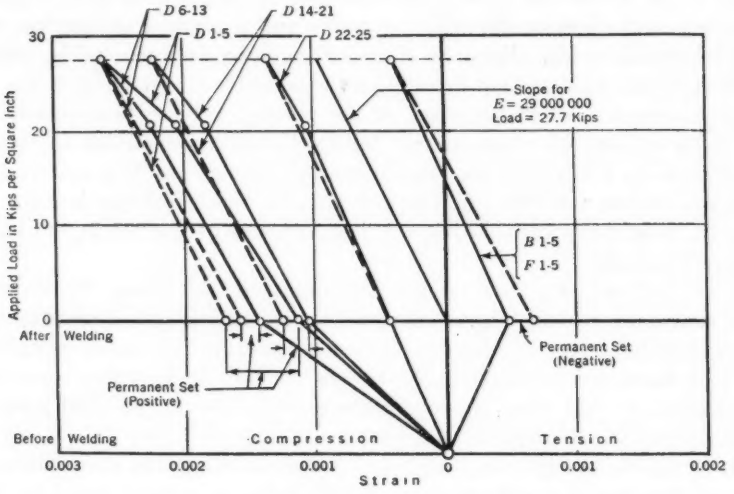


FIG. 17.—LOCAL STRAINS IN COLUMN NO. 8 BASED ON READINGS TAKEN BEFORE FABRICATION.

the strains in the gauge at mid-height (Section *CC*) were weighted in proportion to the approximate areas of the cross-sections which they represented, and weighted average strains for Section *CC* were computed from these weighted values. These strains were plotted in Fig. 10, and from these stress-strain diagrams the moduli of elasticity, given in the last column of Table 4, were computed.

TABLE 4.—MODULUS OF ELASTICITY OF COLUMNS, IN POUNDS PER SQUARE INCH

| Column No. | Modulus from Coupon Tests | | | From column tests |
|---|---|---|---|---|
| | Core section | Plates | Weighted average | |
| 1 | 29 500 000† | 30 000 000† | 29 800 000 | 28 000 000 |
| 1*BB* | 28 400 000† | 30 100 000† | 29 500 000 | 28 400 000 |
| 2 | 31 000 000† | 30 400 000† | 30 600 000 | 28 800 000 |
| 3 | 31 300 000† | 28 900 000† | 29 900 000 | 29 200 000 |
| 4 | 28 000 000† | 27 400 000† | 27 700 000 | 28 400 000 |
| 5 | 27 500 000† | 29 100 000* | 28 300 000 | 29 100 000 |
| Average.... | 29 300 000 | 29 300 000 | 29 300 000 | 28 650 000 |

* Average for four test coupons. † Average for two test coupons.

Only the first five columns tested, Nos. 1*BB*, 2, 3, 4, and 5, were used for these determinations, since in Columns Nos. 1, 6, 7, and 8, some or all of the gauge lines on Section *CC* were omitted. The extreme range in moduli of elasticity from the coupons tested in tension, was about 10% of the average value. The corresponding range of values from the column tests (compression) was about 3 per cent.

The average modulus from the coupons was 29 300 000 and that from the columns, 28 650 000 lb. per sq. in. The agreement between the two values is reasonably good, and their average is very close to 29 000 000 lb. per sq. in., the value used in the computation of stresses from strains in this paper.

The agreement between the weighted moduli for the columns and the coupons is of interest in view of the fact that tests of two independent structures in recent years reported by Clyde T. Morris,[5] M. Am. Soc. C. E., and A. H. Fuller,[6] M. Am. Soc. C. E., indicated that either (1) the modulus of elasticity of the columns was only about 22 300 000 lb. per sq. in.; (2) the load was about 27% greater than that computed; or (3) errors due to temperature deformations entered the data. Both tests appear to have been conducted so carefully, and the results were so consistent for a large number of columns, as to make it unlikely that appreciable error due to temperature changes or inaccuracy in determining loads was involved.

It was largely in an effort to find out whether uneven distribution of stress over the cross-section could explain the discrepancy in the tests by Professors Morris and Fuller that the gauge lines shown in Fig. 6 were distributed so completely over the cross-section of the column. However, the results of these tests offer no explanation of the unexpected phenomena observed by Professors Morris and Fuller.

## 7.—Deflections

The initial alignments of the columns as given by measurements on the cover-plates, are shown in Fig. 2. The load-deflection curves are given in Fig. 18, and the deflections at maximum load in Table 5. In both cases the deflection given occurred in the plane of least resistance.

TABLE 5.—Results of Column Tests

| Column No. | Sectional area, in square inches | $\frac{l}{r}$ | Maximum Load | | Deflection* at maximum load, in inches | Modulus of elasticity, in pounds per square inch |
|---|---|---|---|---|---|---|
| | | | Total per Unit Area | | | |
| | | | In kips | In kips per square inch | | |
| 1 | 24.30 | 80.8 | 785 | 32.3 | 0.68 | 28 000 000 |
| 1*BB* | 24.30 | 80.8 | 784 | 32.3 | 0.33 | 28 400 000 |
| 2 | 24.30 | 80.0 | 790 | 32.5 | 0.50 | 28 800 000 |
| 3 | 23.97 | 77.9 | 739 | 30.8 | 0.75 | 29 200 000 |
| 4 | 14.26 | 78.7 | 527 | 36.9 | 0.77 | 28 400 000 |
| 5 | 14.26 | 78.7 | 510 | 35.8 | 0.87 | 29 100 000 |
| 6 | 23.97 | 77.9 | 799 | 33.3 | 0.62 | 26 000 000 |
| 7 | 14.26 | 78.7 | 523 | 36.7 | 0.38 | 27 500 000 |
| Average..... | ..... | .... | ... | 33.8 | 0.61 | 28 155 000 |

* Deflection measured perpendicular to web.

The following discussion of deflections applies to Column No. 4 which was the first column tested. The behavior indicated here may be taken as more or less typical of all except Column No. 1 which is discussed in Section 8

[5] "Dead Load Stresses in the Columns of a Tall Building," *Bulletin No. 40*, Eng. Experiment Station, Ohio State Univ.

[6] "Measurement of Stresses in Four Steel Columns of the Equitable Building," *Bulletin No. 72*, Eng. Experiment Station, Iowa State Coll.

and except Column No. 8 for which no deflections were measured. Column No. 4 was not entirely straight at the beginning of the test. (See Fig. 2.) Each cover-plate had a camber of 0.08 in. at the center in the plane of the plate, but the camber was in opposite directions for the two plates, so that for deflections of the column as a whole, the effect should be nearly the same as if the column had been straight. The camber in opposite directions might

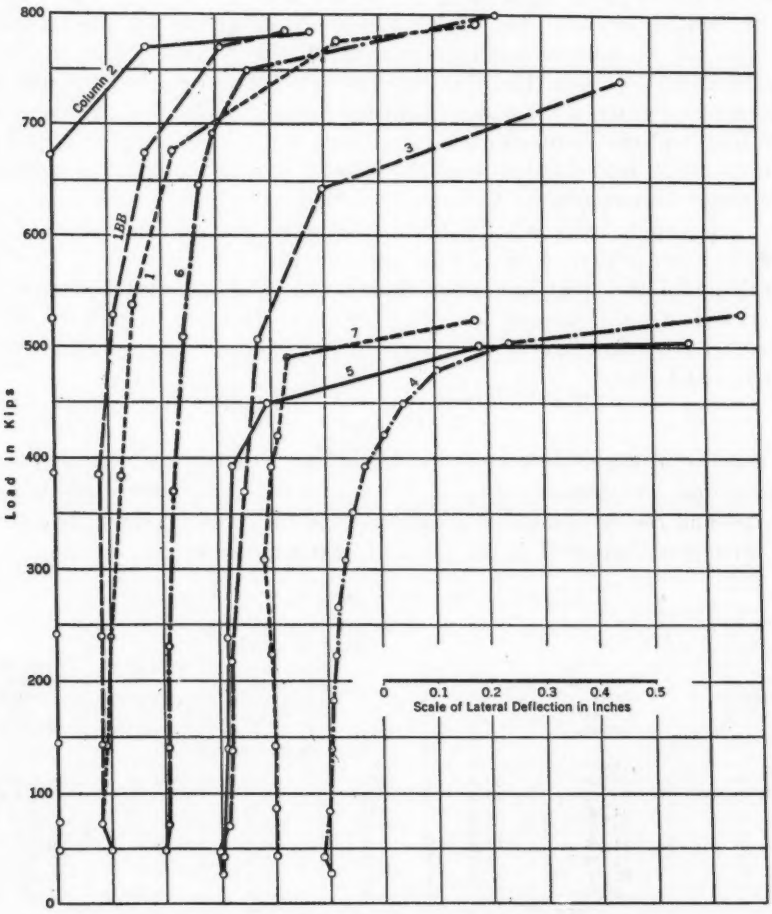


FIG. 18.—CENTER DEFLECTION PERPENDICULAR TO WEB.

tend to introduce a twisting, but if any twist was present it was not large enough to be observed visually.

Some deflection in the plane of least resistance was apparent almost from the beginning of the test (Fig. 18). At a load of 266 kips, this deflection was 0.02 in. and from that point the deflection increased rapidly, becoming 0.77 in. at the maximum load of 527 kips. Deflection in the plane of greatest resistance began at a load of about 266 kips, the load at which the deflection

in the plane of least resistance first began to increase rapidly. From that point, it increased steadily to 0.03 in. at a load of 504 kips, at which time a rapid increase began. At the maximum load of 527 kips, this deflection was 0.06 in. Although the test was continued beyond the maximum load, the instruments had been removed and the deflection was not measured. The principal final deflection took place in the plane of least resistance when, with the testing machine still running, the load had fallen off to about 500 kips. That in the plane of greatest resistance was still not apparent to the eye.

Inspection of Fig. 18 shows that, except at the maximum load, the deflections of the riveted Columns Nos. 2 and 5 were smaller than those of the corresponding Columns Nos. 1 and 4, welded with continuous beads. Fig. 2 indicates that the riveted columns were initially straighter than the continuously welded columns. Fig. 18 shows that Columns Nos. 6 and 7, with intermittent beads, deflected less under load than the corresponding Columns Nos. 3 and 4, with continuous beads. Columns Nos. 6 and 7 were initially slightly straighter than Columns Nos. 3 and 4. Still another comparison may be made. Columns Nos. 4, 5, and 7, were alike except that Column No. 5 was riveted, Column No. 7 was welded with an intermittent bead, and Column No. 4, with a continuous bead. The magnitudes of the initial departures from a straight line increased in the order given—namely, Columns Nos. 5, 7, and 4—whereas the deflection under load was least for Column No. 7 (intermittent weld), next larger for Column No. 5 (riveted), and largest for Column No. 4 (continuous weld).

In all comparable cases the deflections were largest for the columns welded with the continuous bead. Only one comparison between a riveted column and a column with intermittent bead could be made, and the deflection was less for the intermittently welded, than for the riveted, column. The two comparisons available showed the deflection for the intermittently welded, to be less than that for the continuously welded, column.

The columns fabricated by riveting were straighter than those fabricated by either continuous or intermittent welding. The magnitude of deflection under load corresponded to the magnitude of the initial departures from straightness, except that the riveted column (No. 5) gave greater deflection under load than the less straight column (No. 7), which was welded with intermittent beads.

In general, the differences in deflection were small and only one column of a kind was tested. Therefore, it should not be concluded that these statements represent in general the relation between deflections of riveted and welded columns.

### 8.—Bending Stresses

The averages of the measured strains for Gauge Lines 4 and 12, and for Gauge Lines 16 and 24, were computed separately. The former represents the strain on the west, and the latter, the strain on the east, edges of the cover-plates. One-half the sum of these average strains represents the average strain over the entire cross-section and one-half their difference represents the bending strain in the "extreme fiber," provided that (1) the strains were measured correctly; and (2) a plane section before bending remained plane

after bending. These strains, multiplied by 29 000 000 lb. per sq. in. (the average modulus of elasticity for all columns), give the average stress, $\frac{P}{A}$, and the bending stress, respectively.

A comparison of the average stress thus computed, with the total loads divided by the area of the columns—that is, $\frac{P}{A}$ —is given in Fig. 19. The two curves for average stress agree closely for the lower loads. This agree-

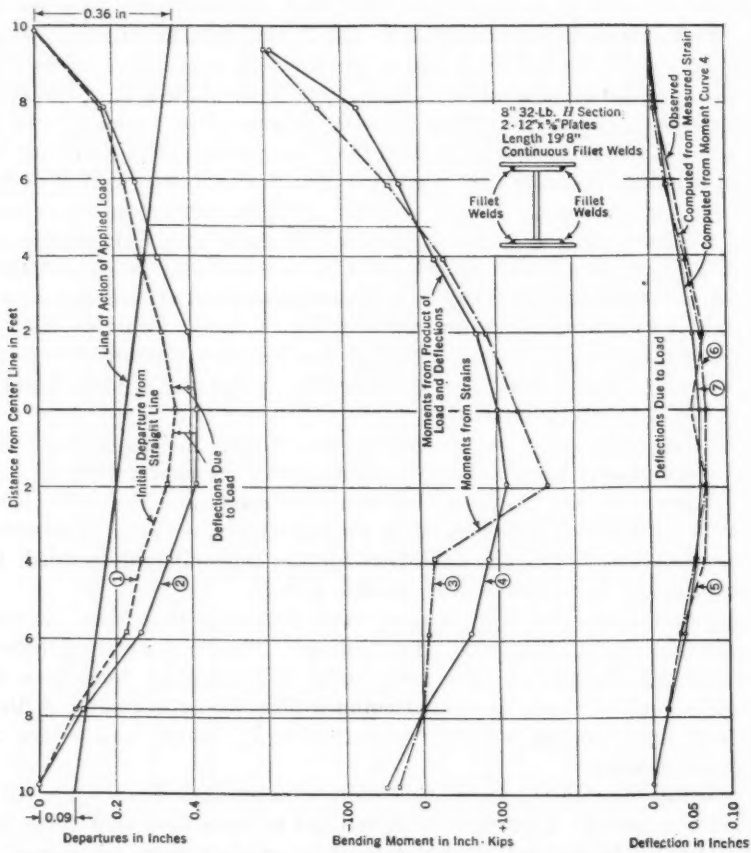


FIG. 19.—LOAD-STRESS CURVES: (*a*) COLUMN NO. 1 *BB*, AND (*b*) COLUMN NO. 2.

ment should be expected up to a point at which the maximum stress is near the yield point of the material, but a marked divergence takes place at a value of $\frac{P}{A}$ = 21 kips per sq. in. for Column No. 1*BB*, and at a value of 16 kips per sq. in. for Column No. 2 (total loads, 510 and 336 kips, respectively), whereas the yield point of the coupons occurred at a stress of about 35 kips per sq. in. for each of these columns (Table 6). It is possible that, due to the presence of initial stresses, the limit of proportionality between

stress and strain had been reached at certain places, under comparatively low average stresses.

TABLE 6.—RESULTS OF COUPON TESTS OF COLUMN MATERIAL

| Column No. | Sectional Area, in Square Inches | | | Yield-Point Stresses, in Kips per Square Inch | | | Stresses at Maximum Load, in Kips per Square Inch | | |
|---|---|---|---|---|---|---|---|---|---|
| | I-section | H-section | Plates | Core, section, I or H | Plates | Weighted average | Core section, I or H | Plates | Weighted average |
| 1 | ..... | 9.3 | 15.0 | 38.8† | 32.5† | 34.9 | 62.2† | 58.7† | 60.1 |
| 1*BB* | ..... | 9.3 | 15.0 | 38.8† | 33.8† | 35.4 | 60.2† | 58.6† | 59.2 |
| 2 | ..... | 9.3 | 15.0 | 40.8† | 32.8† | 35.9 | 61.7† | 57.5† | 59.1 |
| 3 | 10.22 | .... | 13.75 | 35.3* | 34.1† | 34.6 | 56.8* | 58.6† | 57.7 |
| 4 | 7.38 | .... | 6.88 | 38.9* | 38.5† | 38.7 | 59.2* | 59.4† | 59.3 |
| 5 | 7.38 | .... | 6.88 | 40.3* | 38.1‡ | 39.2 | 59.8‡ | 59.8‡ | 59.8 |
| 6 | 10.22 | .... | 13.75 | 40.5* | 36.0† | 37.9 | 66.4* | 65.0† | 65.6 |
| 7 | 7.38 | .... | 6.88 | 39.2* | 40.4† | 39.8 | 62.1* | 57.2† | 59.7 |
| Average..... | ..... | .... | ..... | 39.0 | 35.8 | 37.1 | 61.0 | 59.3 | 60.1 |

* Average of six test coupons. † Average of eight test coupons. ‡ Average of twelve test coupons.

Fig. 8 is a summary of the bending stresses for Columns Nos. 1 to 5, inclusive. The relation between bending stresses at the top, middle, and bottom for Column No. 1 is shown in some detail in Fig. 20. In all cases

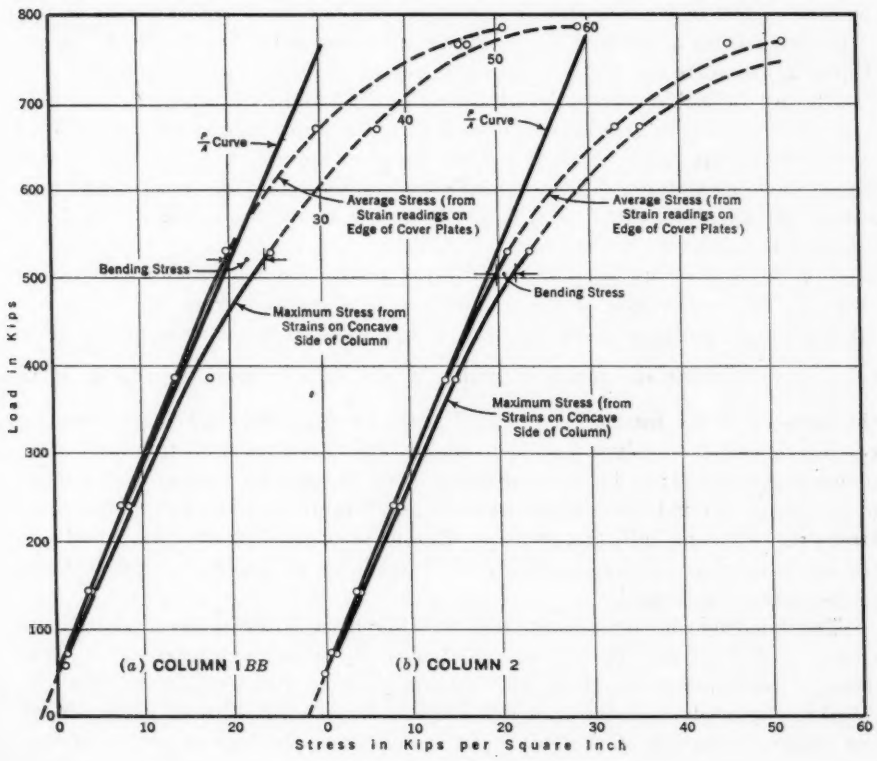


FIG. 20.—LOAD STRESS CURVES.

the bending stresses at the top and bottom, Sections *BB* and *DD*, are opposite in sign from those on the same side of the column at the mid-section, *CC*, although all were plotted in the same direction for convenience. In Columns Nos. 3, 4, and 5, the bending stresses at the top, middle, and bottom were about equal, as would be the case if the columns were fixed at the top and bottom. In Columns Nos. 1, 1*BB*, and 2, the bending stresses were greater at the top than at the middle, indicating some initial eccentricity of loading in the columns. There is nothing in Fig. 8 to indicate any marked advantage of either riveted or welded columns over the other, as far as freedom from bending moments is concerned.

Strangely enough the bending stress seems to have had little effect on the maximum loads carried. This is shown by the fact that the ratio of the stress at maximum load, to the yield-point stress of the material (given in Table 3), was nearly constant for all columns.

### 9.—Bending Moments and Deflections in Column No. 1

In the test on Column No. 1, longitudinal strains were measured on the four corners; that is, on the edges of the cover-plates, at sections approximately 2 ft. apart, throughout the length of the column. At the same sections deflections were measured parallel to, and perpendicular to, the web of the column. The deflections due to a load of 528 kips were added to the initial departures from a straight line and the resulting total departures (Fig. 20, Curve 2), indicate the total curvature in the same plane under this load.

It was desired to ascertain whether the deflections of this column under load were those that should be expected with a column having as much initial curvature as this one possessed. This question appears to be of the greater significance as Column No. 1 carried a maximum load (785 kips) slightly in excess of that (784 kips) carried by the much straighter column, No. 1*BB*, which otherwise was similar to it.

For the load of 528 kips, the bending moments about a gravity axis which was parallel to the web of the columns were computed from the measured strains for all sections at which strain-gauge measurements were taken. For these computations the usual formula, $M = \frac{SI}{c}$, was used, in which, $M$ is the moment of the internal stresses; $S$ is the bending stress; $I$ is the moment of inertia of the column section about the gravity axis to which the moment is referred; and $c$ is the distance from the gravity axis to the extreme fiber, that is, one-half the width of the plate. The strain used in computing the stress, $S$, was one-half the average difference of the strains measured on the opposite edges of the cover-plates. A modulus of elasticity of 29 000 000 lb. per sq. in. was used.

The bending moments computed in this manner have been plotted in Curve 3 of Fig. 20. It will be noted that there were two points of zero moment within the length of the column. Since the bending moment is equal to the product of the load times the distance of its line of action from the center of gravity of the section of the column, the line of action of the load must pass through the center of gravity of the sections of zero bending moment. This fact was used to locate the line of action of the load. Pro-

jecting the two points of zero moment horizontally from Curve 3, to intersect Curve 2, two points on the line of action were established. A line passed through these points indicates that there was an eccentricity of the load equal to 0.36 in. at the top of the column and to 0.09 in. at the bottom. Multiplying the load by the departures of its line of action from the center line of the deflected column gives the moments at the various sections. These moments have been plotted in Curve 4 for comparison with those found from the strains.

While there are certain marked disagreements, the general agreement between these two moment curves, 3 and 4, is good. From each moment curve (that is, that obtained directly from the strains, and that from the product of the load times the departures), the deflections have been determined by a double integration of the $\frac{M}{EI}$-curve for the column. The measured deflections and the deflections computed by the two methods have been plotted in Curves 5, 6, and 7, for comparison with each other. Since the deflection curve could have been determined directly from the measured strains instead of first using the strains to compute the moments, the deflection curve, 6, has been designated as that computed from the measured strain. In determining the deflections given in Curve 7 the moments computed as loads times departures were used. The agreement between the two methods (which is very good), indicates that there was nothing mysterious in the behavior of the column so far as deflections are concerned. It is not clear, however, why Column No. 1 carried a slightly larger load than Column No. 1*BB*, in spite of the lack of alignment of Column No. 1. The explanation is not to be found in a difference of the yield points of the steel for the two columns, since this was only 0.5 kip per sq. in. The weighted average yield point for Column No. 1 was 34.9 kips per sq. in., and that for Column No. 1*BB* was 35.4 kips per sq. in. In spite of its greater initial straightness, the maximum bending stress at the load of 528 kips was greater in Column No. 1*BB* than in Column No. 1, and this may account for the fact that Column No. 1 carried slightly more load than Column No. 1*BB*. Possibly, the lack of planeness of the ends of the columns had more to do with the development of bending stresses than the lack of initial straightness.

It was unexpected to find an eccentricity of 0.36 in. for the resultant load at the top and an eccentricity so much greater there than at the bottom. As indicated in the Appendix, Article 2, much care was used in centering the columns. However, as shown in Fig. 8, bending strains were found early in the tests. Although the column ends were milled, they were not plane enough to provide uniform contact with the bearing-plate, and this lack of planeness seems to be the most probable cause of the eccentricity.

### 10.—Proportional Limit and Maximum Load

The stress at which any curve in Fig. 10 departed appreciably from a straight line was taken as the proportional limit for the column represented by the curve. The values thus determined are given in Table 3. The proportional limits for Columns Nos. 3, 6, and 7, are so low as to raise some

doubt as to the validity of the values selected, but for Column No. 3 the points determined by the strain gauge and those determined by the compressometer readings for the full length of the column coincide throughout the curve, giving confirmation to the value chosen as the proportional limit. For Columns Nos. 1*BB*, 2, 4, and 5, the points determined by the strain gauge coincide with those from the compressometer up to a high load. For Columns Nos. 1, 6, and 7, strain-gauge readings for plotting the average strains were not taken, therefore, no comparison between proportional limit by the two methods was available.

It has been indicated quite clearly that the presence of initial compressive stresses at any position hastened the appearance of strain lines at that position. The stresses at the proportional limit, as shown in Table 3, were higher than the stresses at the first appearance of strain lines, yet there is a general correspondence between the two sets of values which suggests that the presence of initial stresses affected the limit of proportionality between applied stress and resulting strain.

Table 3 gives a comparison of the strengths computed by the use of the Rankine formula,

$$\frac{P}{A} = \frac{S}{1 + \frac{1}{25\,000}\frac{l^2}{r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

with the stresses at maximum load. In this formula: $P$ = maximum load; $A$ = sectional area; $S$ = yield-point stress from the coupon tests; $\frac{l}{r}$ = slenderness ratio; and, $\frac{1}{25\,000}$ = an empirical constant for steel columns with fixed ends. The Rankine formula was used for comparison, because it is well known and widely accepted.

The strengths of different columns cannot be properly compared merely on the basis of the stresses developed at the maximum load. It has been found[7] that for columns of the same slenderness ratio, $\frac{l}{r}$, the stresses at maximum load were approximately proportional to the yield-point stress of the steel from which the columns were made. The ratio of the average stress at maximum load on the column to the yield-point stress of the steel thus becomes a measure of the effectiveness with which the material in the column is used. The effectiveness ratio, that is, $\frac{\text{yield point}}{\text{column strength}}$, has been computed and is given in the next to the last column of Table 3. The highest ratio given is 0.954; the lowest is 0.897; and the average is 0.913. The maximum variation from the average is 4.4 per cent. The average effectiveness ratios for the riveted, continuously welded, and stitch-welded columns were 0.909, 0.921, and 0.900, respectively.

[7] *Technologic Paper No. 328*, Bureau of Standards, p. 85.

On the whole, it must be concluded that the maximum loads stated in terms of the yield-point stress of the material determined at speed of testing stated, that is, the effectiveness ratios, were so nearly equal regardless of the method of fabrication of the columns, that no reliable distinction can be made between the merits of the three methods represented.

The ratio of the strength, $\frac{P}{A}$, computed by the Rankine formula, to the average stress at maximum load, is given in the last column of Table 3. The values of this ratio range from 84 to 92%, and the average is 88% of the maximum load.

## 11.—Summary of Results

The results of this investigation may be summarized in the following eleven items:

(1) The largest slipping of plates shown by the measurements, was about 0.001 in. Even this may have been merely a difference in compressive deformation within the gauge length over which the measurements were taken. No weaknesses attributable to slipping of plates developed.

(2) Stitch welding caused shortening of the metal at sections through the welds and elongation at the edges of the cover-plates at sections midway between welds. In the stitch-welded columns, the strain lines appeared first on the sections through the welds, but none appeared in the welds or in the metal very close to the welds. In general, they did not appear between the welds except in the webs of the columns, where their formation apparently was brought about by initial compressive stresses set up in the cooling of the section after rolling.

(3) The points of first appearance of strain lines corresponded in position to the points of highest stress due to heating of the metal, as far as the stress due to heating could be determined.

(4) The average modulus of elasticity determined from the coupon tests was 29 300 000 lb. per sq. in., and that determined in the tests of the columns was 28 650 00 lb. per sq. in. The modulus used in computing stress from strain was 29 000 000 lb. per sq. in.

(5) In all comparable cases, the deflections were greatest for the continuously welded columns. There was little distinction as to deflection between the riveted and the stitch-welded columns. In general, the magnitude of the deflection corresponded to the magnitude of the initial departures from straightness. However, only one column of a kind was tested, and the differences in deflection are not large. Therefore, conclusions as to deflection should be used with caution.

(6) In Columns Nos. 3, 4, and 5, the bending stresses, and, therefore, the bending moments, were about equal at top, middle, and bottom. In the other columns, the position of maximum moment varied between top, middle, and bottom. Nothing in the tests indicated any marked advantage of either riveted or welded columns over the other as far as freedom from bending

moments is concerned, although the continuously welded columns seemed to be more subject than the others to initial deflection.

(7) In spite of its large initial deflection, Column No. 1, which was continuously welded, carried slightly greater maximum load than its counterpart, Column No. 1*BB*. The bending moments in this column, computed as the product of the loads times the deflections relative to the line of action, agreed fairly well with the moments computed from the measured strains. The deflections computed from the bending moments also agreed very well with the measured deflections.

(8) The limit of proportionality between applied stress and resulting strain occurred at stresses well below the yield point, but higher than the stresses at which strain lines first appeared. There was, however, a general correspondence between the stress at the limit of proportionality and that at which strain lines first appeared, and this suggests that both were influenced by the presence of initial stresses in the columns.

(9) The ratios of the stress at maximum load to the yield-point stress determined from the coupons were so nearly equal for all the columns tested, that no reliable distinction can be made between the merits of riveting, or welding, columns, either with continuous or an intermittent weld.

(10) Although the tests indicated the presence of initial stresses of considerable magnitude, introduced in the cooling after welding (and probably after rolling also), the maximum loads carried did not appear to be appreciably influenced thereby.

(11) The average stress at maximum load was about 15% greater than the strength computed by the Rankine formula, Equation (1).

### Acknowledgment

This investigation was made at the Fritz Engineering Laboratory of Lehigh University for the McClintic-Marshall Company. Acknowledgment is made to that Company for permission to use the results in this paper and personally to Mr. G. L. Taylor and Jonathan Jones, M. Am. Soc. C. E., successively, Chief Engineer, for personal assistance and advice. The writers wish also to thank the following men then of the Department of Civil Engineering: R. J. Fogg, M. Am. Soc. C. E., for helpful suggestions; C. D. Jensen, Assoc. M. Am. Soc. C. E., for carrying out the test of the short column discussed in Section 5, and for furnishing interpretations of this and other parts of the investigation; and Mr. R. B. Morris, now of the Port of New York Authority, for valuable assistance in testing, and in compiling and interpreting the data.

## APPENDIX

*1.—Test Coupons.*—A piece 1 ft. long was cut from each rolled section (plates, I-beams, and H-beams) before the columns were shipped from the fabricating shop. From this piece, four test coupons were taken for each cover-plate, six for each I-section, and eight for each H-section. These coupons were tested in tension for yield point and ultimate strength, and a

few for modulus of elasticity. Hence, the yield point and ultimate strength of the steel reported for each column is based on the average of fourteen or sixteen coupons. As a result of this procedure the columns were 1 ft. shorter than the lengths originally designated.

Fig. 21 shows by cross-hatching the location and numbering of the test coupons cut from the various sections. The letter, *A*, when present, indicates that the coupon was taken from the core, that is, from the I-section or the

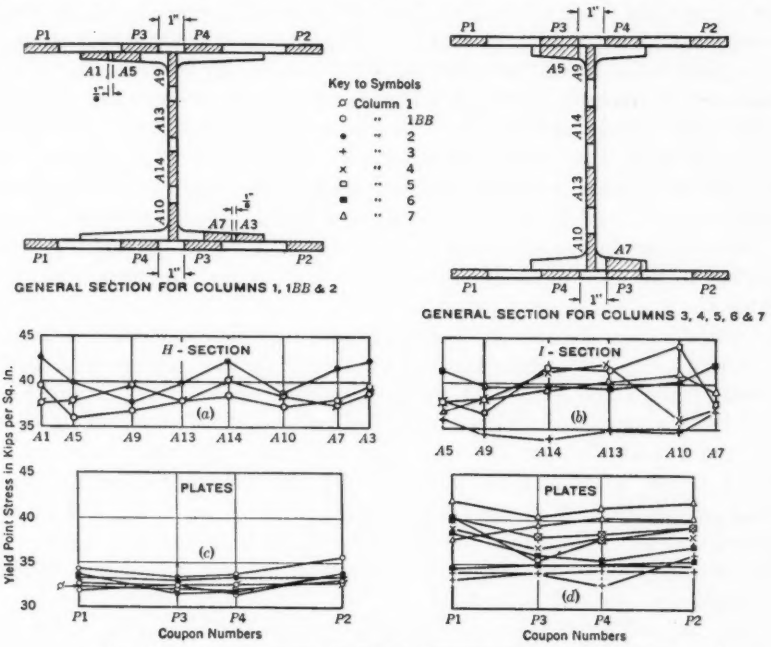


FIG. 21.—YIELD-POINT STRESSES DETERMINED FOR COUPONS.

H-section. The letter, *P*, indicates that it was taken from one of the plates. The final numeral indicates the location in the section. The coupons were arranged with a view of determining whether there were systematic variations of yield point throughout the section. All specimens were $1\frac{1}{2}$ in. wide, except *A*1, *A*3, *A*5, and *A*7; and Columns Nos. 1, 1*BB*, and 2, which were $1\frac{1}{4}$ in. wide.

The tests of the coupons were made in a 50 000-lb. testing machine in the usual manner for tension tests. The yield-point load was determined by the "drop" of the weighing beam in the tests in which the modulus of elasticity was not determined. In the modulus tests, the yield point was chosen as the load for which the strain indicated by the gauges showed a rapid increase with little increase in load. An extensometer with a gauge length of 2 in. was used in the tests for modulus of elasticity.

Considerable attention has been paid to the determination and weighting of yield-point stresses, because the strength of the column has been found to be more directly a function of the yield point than of the ultimate strength of the materials from which the column was made.

The yield-point stresses for individual coupons have been plotted in Fig. 21. The horizontal distances between points are proportional to the distances from center to center of coupons, so that a horizontal line on the diagram representing the average yield point for all the coupons for a given plate, will be weighted approximately according to the sectional areas represented by the several coupons. Fig. 21 (*a*) shows a tendency for the higher yield points to occur at the edges of the flanges and at the center of the web, and a lower yield point at the roots and at intermediate points on the flanges. However, examination of Fig. 21 (*a*) to Fig. 21 (*d*) indicates that the variation in yield-point stress for different locations in any section (either plate or core section) from the average, was not greater than about 2 kips per sq. in. An average not weighted for the area represented by individual coupons was used, therefore, as the yield-point stress for each section. The average yield points for the plates, however, differed in general from those for the corresponding cores, and it was necessary, in determining the yield-point stress for a column, to weight the yield point of the plates and that of the core in proportion to the area of each. This has been done, and the results are given in Table 6.

The yield points for the three **H**-sections show little variation from each other, while the yield points for the **I**-sections showed a somewhat greater variation, due to the low yield point for the **I**-section in Column No. 3. Among the plates the variation of yield point was greatest of all. Columns Nos. 6 and 7 were furnished considerably later than the other columns, and the yield points of the plates in these columns were considerably greater than the average for all plates. With only one exception, the yield point of the plates was lower than that for the core section in the same column. The maximum variation of an individual from the average of all the weighted yield points was about 7 per cent.

*2.—Testing.*—The following description of methods applies to Column No. 4 which was the first column tested. The others were tested in the same manner except for minor changes and improvements as follows:

(*a*) Two-inch strain-gauge readings (Section *AA*, Fig. 6) were omitted from the other column tests.
(*b*) Four more gauge lines at Section *DD* in which slipping of the plates was measured, were added in the later tests.
(*c*) Gauge Lines 1, 3, 13, and 15 were added at Sections *BB* and *DD*.
(*d*) The testing of Column No. 1 differed materially from that of the other columns. This test is described in Section 9 of the main body of this paper.

The locations of the gauge lines on the cross-section of Column No. 4 are shown in Fig. 6. The gauge length at Section *AA* was 2 in.; at Sections *BB*, *CC*, and *DD*, it was 10 in. The gauge holes were drilled with a No. 54 drill (0.054 in. in diameter). Gauge Lines 5, 11, 17, and 23 in Section *BB*, were on the edge of the **I**-beam which formed the core of the column. Gauge Lines 5*b*, 11*b*, 17*b*, and 23*b*, had the same upper holes as Gauge Lines 5, 11, 17, and 23, but the lower hole for each of these lines, instead of being in the **I**-beam, was in the cover-plate close to the fillet weld. Equality of strain in

Gauge Line 5 with that in Gauge Line 5*b*, would indicate that there was no slipping of the plate, but any difference in the strains between Gauge Lines 5 and 5*b* indicates slipping of the plate relative to the I-beam, and this is true also with the other similarly placed gauge lines.

Before placing Column No. 4 in the testing machine, it was whitewashed with a mixture of white Portland cement and water, in order to make more evident the flaking of the mill scale (herein termed "strain lines") as the column came under stress.

After it had been placed in the machine, and before any load had been applied, provision was made for measuring the shortening over the entire length of the column at its four corners, and for measuring the center deflection relative to the ends. The instruments for measuring this shortening are here termed "compressometers" to distinguish them from the strain gauge. Each compressometer consisted of an Ames gauge and a distance piece. The distance piece was a $\frac{3}{8}$-in. steel pipe attached to the column 1 in. below the top, and of such length that its lower end bore against the plunger of the Ames gauge fastened 1 in. above the bottom of the column. Thus, the compressometers measured the total shortening of a column under load, except that which occurred within the 2 in. outside the gauge lenth. The distance piece was held in line by guides at about the one-third points of the length of the column. The Ames gauges read satisfactorily to thousandths of an inch. Each 0.001-in. total shortening corresponds to an average strain of $\frac{0.001}{182}$, or 0.0000055 in. per in. for the 182-in. gauge length, or to an average stress of 160 lb. per sq. in., using a modulus of elasticity of 29 000 000 lb. per sq. in.

For measuring the deflections, two fine wires were attached to the web of the column 6 in. below the top and 6 in. above the bottom, giving a span of 172 in. The deflection was read to the nearest 0.01 in. on scales attached to the column. The arrangement is shown in Fig. 22.

The column was tested in a standard two-screw testing machine of 800 kips capacity. As placed in the machine, the web of the column was in the plane of the longitudinal center lines of the screws. The bottom of the column was placed on a planed steel plate 3 in. thick, which rested directly on the bed of the machine.

This column was placed so that its top was centered accurately between the screws, letting the bottom of the column come where it would. Mounted on top of the column was a 3-in. plate, planed on both sides. On top of the plate and directly under the head of the compression block, were three pairs of wedges, each resting on a spherical bearing block. The three spherical bearing blocks, as placed on the 3-in. plates, outline an isosceles triangle, the apex block being centered over the web, $4\frac{3}{8}$ in. north of the center line of the column. The other two bearing blocks were $4\frac{5}{8}$ in. south of the center line of the column, 8 in. apart, center to center.

The compression head of the testing machine was then brought down until the wedges were nearly in contact with it; the wedges were driven

in until they were equally tight as nearly as this could be judged; a load of 15 kips was applied to the column; and the compressometer readings were taken on all four edges of the column. This was followed by a load of 60 kips after which the compressometers were again read. If the difference in

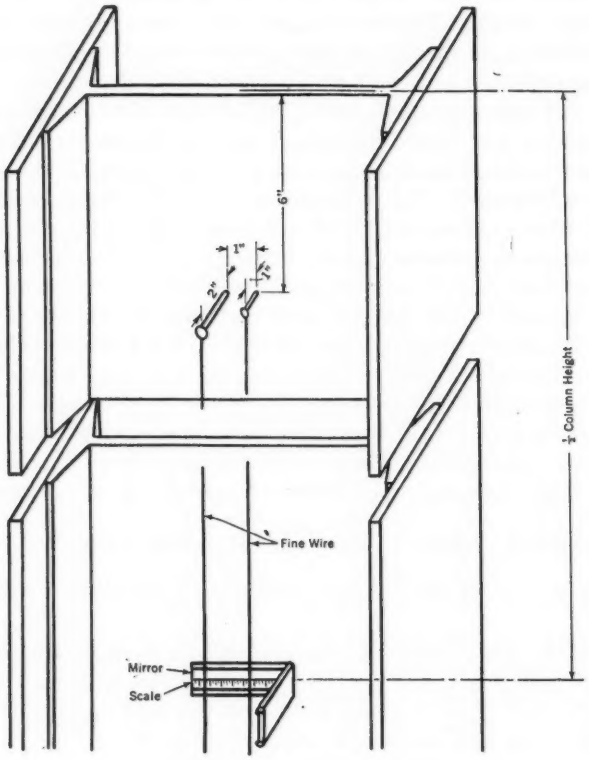


FIG. 22.—DEFLECTION APPARATUS.

shortening on the four edges of the column revealed an eccentric loading, the compression head was raised, and the wedges were re-adjusted. This procedure was repeated until the shortenings measured by the four compressometers were equal within 0.002 in. in the 182-in. gauge length. The loading was then considered to be nearly enough axial to permit proceeding with the test.

The test was carried out with the increments of load shown in Fig. 18. Readings were taken on the compressometers and the deflection apparatus at more frequent intervals than on the strain gauges.

In the case of Column No. 4, after reaching the maximum load the operation of the testing machine was continued until the load had decreased to about 500 kips. Column No. 8 was not loaded to its maximum. In all other cases the loading was discontinued as soon as it was evident that the maximum load had been reached.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

# THE MARTINEZ-BENICIA BRIDGE[1]

By W. H. Kirkbride,[2] M. Am. Soc. C. E.

Synopsis

This paper contains a description of a double-track railroad bridge across Suisun Bay, California, constructed by the Southern Pacific Company. It replaces train-ferry operations across the Carquinez Strait between Benicia and Port Costa, Calif. The project includes other steel structures, in addition to the main bridge, and about 20 miles of railroad track equipped with signal and interlocking plants, on which track provision is made for grade separations for trains, and likewise for vehicular traffic.

## Geology

Thirty-six miles in a northeastern direction from San Francisco, Calif., near the upper end of San Francisco Bay and connected to it by a narrow strip of water known as Carquinez Strait, is a body of water about 31 000 acres in extent. It is formed by the united waters of the Sacramento and San Joaquin Rivers flowing westward toward the Pacific Ocean and being retarded in their course by a range of mountains known as the Coast Range. The Carquinez Strait forms a connecting link by which the rivers unite with San Francisco Bay (see Fig 1). This inland body of water is known as Suisun Bay and, in former ages, covered a much larger area. After filling in the large inland areas now known as the Sacramento and San Joaquin Valleys, the two silt-carrying rivers began to deposit their sediments against the Coast Range, thereby reducing the area of this ancient sea and building up adjacent alluvial lands which are not equalled in fertility

In some cases inland bays along the Suisun shore line were filled with sediments containing a large percentage of vegetable matter. These bays are now virtually peat bogs, unstable in character. Such material is also found on the bottom of Suisun Bay.

Note.—Discussion of this paper will be closed in December, 1932, *Proceedings*.

[1] Presented at the meeting of the Structural Division, Sacramento, Calif., April 24, 1930.

[2] Chf. Engr., Southern Pacific Co., Pacific Lines, and Northwestern Pacific R. R., San Francisco, Calif.

Geologically, some interesting events have occurred in this vicinity. Evidence is found of former recurrent elevation and subsidence of the surface; pronounced earthquake faults exist upon which adjacent blocks of the earth's crust have moved in a northern and southern direction; and tremendous resistance to such movements has resulted in folds and distortion of the rock strata.

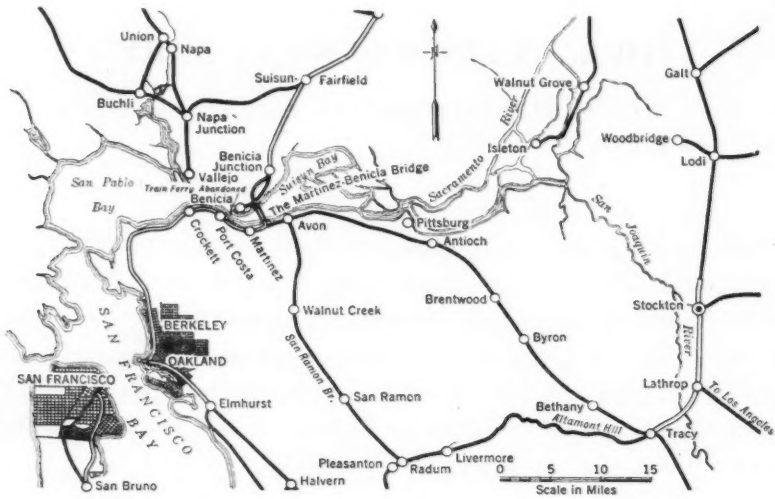


FIG. 1.—MAP SHOWING LOCATION OF THE MARTINEZ-BENICIA BRIDGE.

The hillsides bordering along the Carquinez Strait are broken and disintegrated to such an extent as to be very unstable, and slides are frequent, entailing unusual care and extra construction on the part of those engaged in the building and maintenance of railroads and highways in this territory.

## EARLY RAILROAD DEVELOPMENT

The first railroads in California were constructed out of Sacramento, the first line being to Folsom. This was followed by the completion, in May, 1869, of the famous Central Pacific Railroad that formed a link in the first transcontinental line.

During the construction of the Central Pacific, a railroad was built in 1868 from South Vallejo to Sacramento, *via* Suisun, passing over a spur of the Coast Range and having a summit elevation of 315 ft. above sea level. In connection with this railroad a steamer line was operated from South Vallejo to San Francisco.

Next followed the construction, in 1869, of a railroad from Sacramento to Oakland, *via* Tracy, passing over the Altamont Summit of the Coast Range at Elevation 740 above sea level.

The railroad from Oakland to Martinez, along the southern shore of San Francisco Bay and the Carquinez Strait, was placed in operation in January, 1878, and, in the latter part of 1879, was also extended to Tracy. Thereupon,

the steamer line to San Francisco was discontinued, and a ferry was established across the Carquinez Strait, connecting South Vallejo with the Tracy-Martinez-Oakland Line.

In December, 1879, a more direct low-level route between San Francisco and Sacramento being desired, the railroad passing through Suisun was projected directly across the marsh and peat lands on the north shore of Suisun Bay to Benicia.

A bridge across the Carquinez Strait or Suisun Bay was considered, but was found not economically feasible, and, therefore, a train-ferry system was established by which passenger and freight trains were transported from Benicia to a connection with the railroad on the south shore, previously referred to as the Tracy-Martinez-Oakland Line. This was accomplished by the construction of docks and car-ferry slips at Port Costa and also at Benicia, and by building the steamer, *Solano,* which remained in service until 1930 after fifty-one years of operation.

It is interesting to note that the designer of this boat made use of the principle of the truss bridge in designing the hull. In 1879, trains were small and rolling equipment was light; but notwithstanding this the *Solano* remained the largest car-ferry in the world up to the time it was removed from service. In 1914, as part of a second-track program, double-track car-ferry slips were constructed, and a second car-ferry, the *Contra Costa,* was put into commission.

It is interesting to note that the docks and slips were maintained with untreated piling from 1879 to 1919, a long life indeed for such original construction. However, in 1919, an infestation of marine borers (*teredo navalis*) destroyed the piling, and since that time creosoted piling has been used. The presence of the marine borers was due to the encroachment of salt water up the rivers, which, in turn, was caused by the use of fresh water for irrigation and by a long series of years in which the rainfall was sub-normal, thereby reducing the flow of fresh water through the strait.

The bridging of Suisun Bay or Carquinez Strait had long been under consideration. In the late Eighties newspaper discussions of the project were published and the Army Point-Suisun Point alignment was seriously considered by railroad officials. Later, a prominent bridge engineering company was requested to make studies, and following this, in 1904, during the time of the late E. H. Harriman, the late William Hood, M. Am. Soc. C. E., prepared plans for a bridge to replace the ferry system. Sundry locations were investigated, including one some distance up stream from the Army Point-Suisun Point alignment, and also several in the vicinity of Benicia and Port Costa, and others as far west as Vallejo Junction.

A series of test piles driven across the Bay showed that the bottom was of soft material, and that the depth to hardpan and rock strata was great; but it gave no clue as to the exact character of the bed-rock. It was decided to build a low-level bridge consisting of a swing bridge with a certain number of fixed spans and ballast deck trestle approaches. However, the application was denied by the War Department and the project was then dropped for the

time being. Subsequently, the bridge studies were resumed and a crossing near Antioch was considered in connection with the extension of the Walnut Grove Line, but this route was rejected.

All routes crossing the Carquinez Strait proper were then likewise rejected because of the broken formation, slips in the hillsides, the nature of the terrain, the great depth of water, and, most important, the unfortunate necessity of crossing a well-established earthquake fault known as the "Southhampton."

Finally, in 1927, traffic conditions became acute and the Company faced the necessity of replacing the two existing car-ferries and buying a third, or of building a bridge. The bridge was then an economic necessity.

The two solid rocky points of land—one known as Army Point on the north shore of Suisun Bay, two miles east of Benicia, and the other as Suisun or Bulls Head Point near Martinez and adjacent to the tracks of the Tracy-Martinez-Oakland Line—offered the necessary supporting ground for the approaches and abutments, and permitted the solution of the railroad problem of providing connecting lines.

The Government demands a navigable channel 300 ft. wide for deep-draft vessels near the Martinez shore, but small craft, including fishing boats, use the entire width of the Bay. The presence of considerable soft mud on the bottom was known and the existence of hard rock below the mud was surmised; but the nature, extent, and depth below the water surface of such rock was not known. Proper data of the geological and earthquake fault condition were not of record. Accordingly, after a preliminary survey and a paper location of the route had been made, the services of geologists of experience and reputation were secured to report on the geological features, particularly

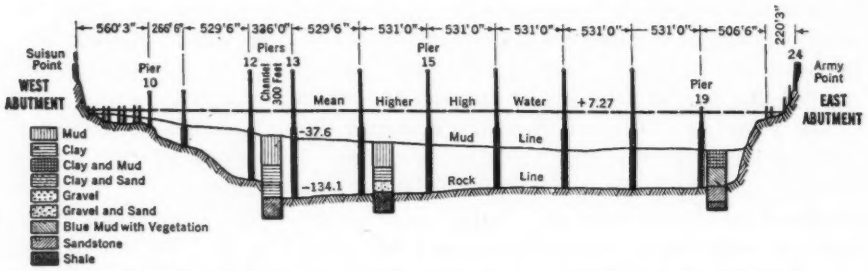


FIG. 2.—CROSS-SECTION OF SUISUN BAY ALONG CENTER LINE OF BRIDGE.

with reference to the earthquake faults. This was followed by an exact determination of bed-rock conditions. For such purpose a row of diamond drill holes was put down across the Bay on the alignment of the bridge, as then conceived.

Bed-rock, consisting of alternate layers of shale and sand stone tilted up on edge at an angle ranging from 60° to 70° from the horizontal, was found at varying depths extending across the channel (see Fig. 2). Drill holes were carried well below the rock surface and the cores were carefully preserved and recorded.

A standard well rig, supported on a 14-pile staging, was used in drilling through the mud. A steel frame of angle irons was lowered through the water to the mud surface and through this a 10-in. well-casing extended to bed-rock, after which the diamond drill was used to explore the rock. A 2-in. rock core was secured. The nature of the mud and gravel overlying the rock was studied, and bearing values were determined. It was thus concluded that the bridge piers should be carried to, and founded on, the bed-rock.

The depth below low water to bed-rock ranges from 115 ft. near Army Point to 136 ft. near Suisun Point, at the location of the lift span. From these depths the rock slopes rapidly upward to the shore line on both sides. The depth of mud near Army Point is 59 ft., while the greatest depth of 101 ft. is south of Pier No. 13. The greatest depth of water is 57 ft. on the Army Point side. It is interesting to note that before the deposition of the present sediments the eroded channel was on what is now the shallow side.

The geologists reported the existence of two well-established major faults extending, parallel to each other, in a northwestern and southeastern direction. The "Southhampton" Fault is west of the bridge and passes under the ferry lanes between Benicia and Port Costa, and the "Green Valley" Fault is east of the bridge, 5 or 6 miles away.

In the Martinez hills various fault lines run in a direction angular to the major faults. The dip of the strata showed a reversal of direction, thereby indicating a folding of the earth's crust, local in extent, and also the development of a syncline. The direction of one of these fault lines was such as to present a serious question. Did it pass under the bridge? Unfortunately, the sediments in the Bay hid its true direction.

While there was some difference of opinion on the part of the geologists, they all agreed that the fault was not active and that any future movement of adjacent blocks along this fault line would be nominal. Its projection on a map shows that it might pass across the bridge tangent near the Army Point shore. On the other hand, the evidence showed that the fault might be a local shear or break in the formation caused by the folding of the strata, but which might not extend far into the formation upon which the bridge was located. This formation on the east was classified as "Chico-Cretaceous" and bedded against it on the west was a formation known as "Martinez," and bedded on the "Martinez" was a formation known as "Tejon." The folded and distorted strata seemed to be confined to the "Martinez" and "Tejon" formations.

However, if the fault line did extend beyond Army Point, it became important to determine its exact location, or at least to avoid the possibility of the most northerly deep-water pier (No. 19, Fig. 2) being placed on it. It was evident from the geological data developed that any slippage along the so-called Martinez Fault would cause the "Martinez" formation to be projected northward into and across the alignment of the bridge. It was decided, therefore, to put down additional diamond drill holes on the site of Pier No. 19. These core drillings, as well as those of all other diamond drill holes, were carefully studied by the geologists and paleontologists. Microscopic and other studies were made to determine from the fossils whether there had been

any change in the rock formation. No such change was found, and the conclusion followed that the Martinez Fault did not extend across the bridge location and that the bridge foundations would rest solely on one monolithic block of geological formation, namely, the "Chico-Cretaceous."

However, for other obvious reasons, it was decided to design the bridge piers to resist earthquake vibration. Accordingly, they were designed with centers of gravity of mass as low as possible. Concrete specified for this purpose had a compressive strength of 2 500 lb. per sq. in., and a tensile strength of 400 lb. at the end of 28 days. It was further specified that the piers should actually be founded on bed-rock and that they should have 30 lb. per cu. yd. of steel reinforcement, placed so as to withstand earthquake shocks of an acceleration of 5 ft. per sec. per sec., which is twice the intensity of the San Francisco earthquake of 1906, and 1.5 times the intensity of the Tokyo, Japan, earthquake of 1923.

The superstructure was designed to resist similar shocks, the tops of the piers being widened and the base of the steel bridge seats being notched into the concrete in order to prevent the spans being pulled off their piers. In designing a pier structure of this kind to resist earthquake vibrations, a quick reduction from a large mass to a smaller mass is a source of weakness; likewise, a construction joint is undesirable and monolithic construction is essen-

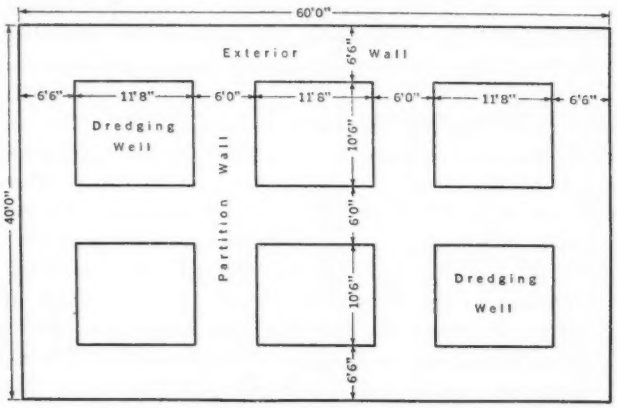


FIG. 3.—DIMENSIONS OF PIER NO. 13, MARTINEZ-BENICIA BRIDGE.

tial. To secure this, as between caisson and pier shaft, and to overcome the difference of inertia of the two different masses, reinforcement rods extend upward from the outer walls of the caisson, pass through the distributing block, and curve inward and upward into the shaft of the pier (see Fig. 3).

## BRIDGE FOUNDATIONS

The substructure consists of two U-abutments, eight viaduct pedestal piers on land, and fourteen viaduct pedestal piers and ten truss span piers in water. Of the ten piers in water, eight are classified as deep-water piers, inasmuch as the rock bottom upon which they are founded ranges in depth below low water from 115 to 134 ft.

Pier No. 11 represents the shallow-water type that can be built by the open coffer-dam method. The size of the concrete base from which the pier shaft rises is 38 by 60 ft. It is oval in shape, with the top of the coping 16 by 49 ft., the side batter 24 on 1, and it was constructed within a steel sheet-pile coffer-dam, rectangular in shape.

Pier No. 13 is the deepest of the deep-water piers. The caisson for this pier was 40 to 60 ft. in cross-section and contained six dredging wells,

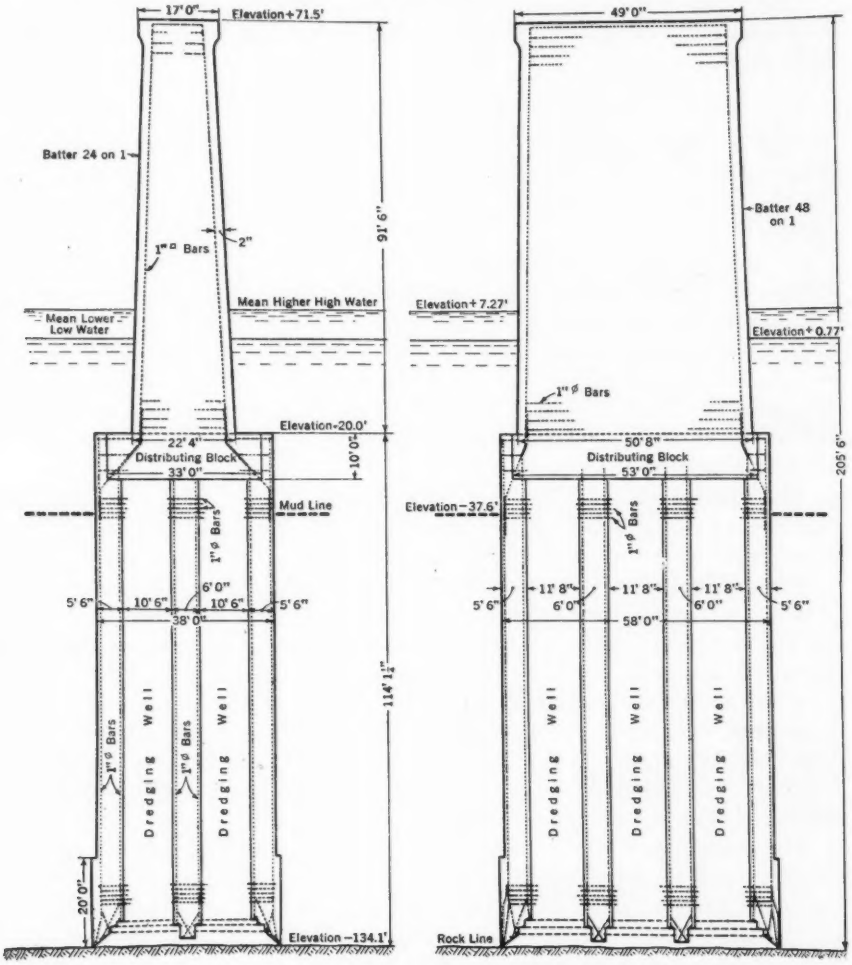


FIG. 4.—CROSS-SECTION OF CAISSON, PIER NO. 13.

10.5 by 11.7 ft. in cross-section (see Fig. 4). The exterior walls were 6.5 ft., and the partition walls 6 ft. thick. The bottom of the caisson was of structural steel, shaped on the outside into a sharp cutting-edge, the interior partition struts conforming to the walls of the six dredging wells, being blunt on the lower surface to lend stability to the caisson during sinking operations

and to distribute the foundation reactions better. Except for the dredging wells, the caisson distributing block and pier shaft were of reinforced concrete. The dredging wells were filled with tremie concrete. The total volume of concrete in this pier is 12 283 cu. yd. The total weight of structural and reinforcing steel is 444 520 lb. Pier No. 12 (Fig. 5) is identical in design, except as to height. These two piers support the lift span and towers, and the adjacent spans.

The other deep-water piers are similar in design, except that the caisson dimensions are 38 by 60 ft. and the dimensions on top of the coping are 16 by 49 ft.

## Construction

The construction of the part of the substructure on land followed the usual method of excavating to suitable rock foundation, placing forms, and pouring concrete; and the viaduct pedestals in water were constructed by driving coffer-dams of sheet-piling.

Piers Nos. 10 and 11 were constructed by the open coffer-dam method. Interlocking steel piles, 65 ft. long, were driven within timber falsework of piling on 7-ft. centers. Battered brace piles also were used, with two rows of 12 by 12-in. timber wales bolted to them. A single row only of sheet-piling was used in Pier No. 11. The inside bracing consisted of a crib or grillage of large, full length timbers, formed of 12 by 12-in. struts, 12 by 12-in. posts, and 12 by 24-in. wales tied together by 3 by 10-in. diagonal bracing, all securely bolted together (see Figs. 6 and 7).

The grillage was sunk nearly to bed-rock by being loaded with about 270 tons of steel rails. Mud and clay were dredged by means of a floating derrick barge, from which a clamshell bucket was operated.

When shale, sandstone, and bed-rock was exposed, about $1\frac{1}{2}$ ft. of it was removed. The surface was then flushed clean, and concrete was poured "in the dry," the forms first having been set up inside the wales and sheet-piling. A mat 6 ft. thick was thus poured, after which the forms were set in to the prescribed section of 58 by 38 ft. and carried to Elevation — 36, where they were again set in to 53.4 by 22 ft. and carried to Elevation — 8. Above this elevation a pier of prescribed dimensions was constructed to bridge-seat elevation. One blow-out occurred while the coffer-dam was being unwatered, but the contractor overcame the difficulty without undue delay.

The construction of the deep-water piers presented a difficult problem. The depth of the water prevented open coffer-dam work, or the use of the pneumatic process. It was proposed, therefore, to sink caissons by the open-well dredging method. The depth of water, the existence of a maximum tidal fluctuation of about 10 ft., the swiftness of the current during the winter floods and at rip tide, and the softness of the underlying bay deposits, all made the launching and sinking of a timber crib difficult.

The contractors, to whom the foundation contract was awarded, proposed a unique method of sinking these deep-water caissons. The accepted construction involved, first, the driving of piles to support an octagonal-shaped platform. Upon this platform eight gallows frames equipped with hand

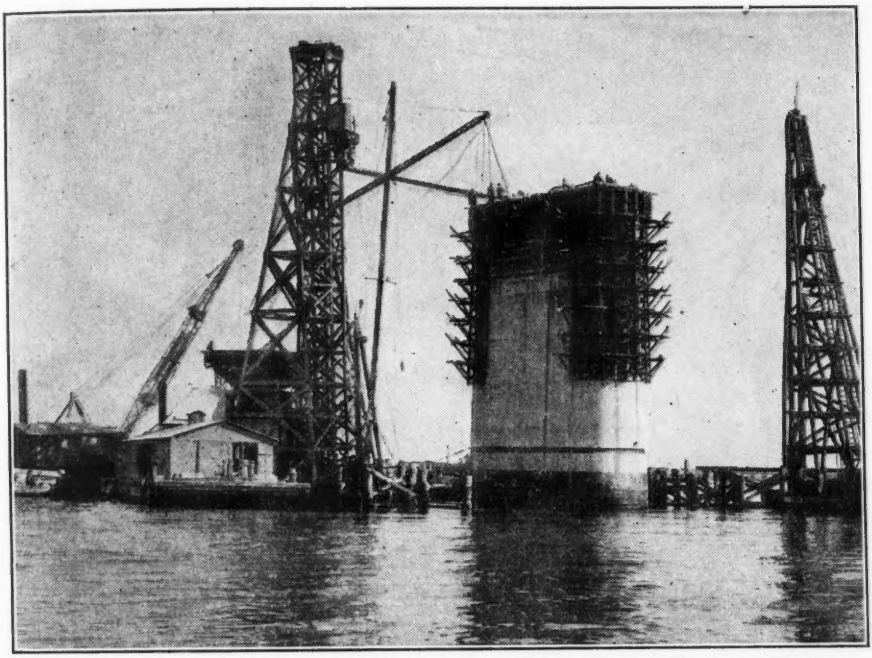

FIG. 5.—PIER NO. 12, MARTINEZ-BENICIA BRIDGE, FORMS SET TO FINISHED HEIGHT.

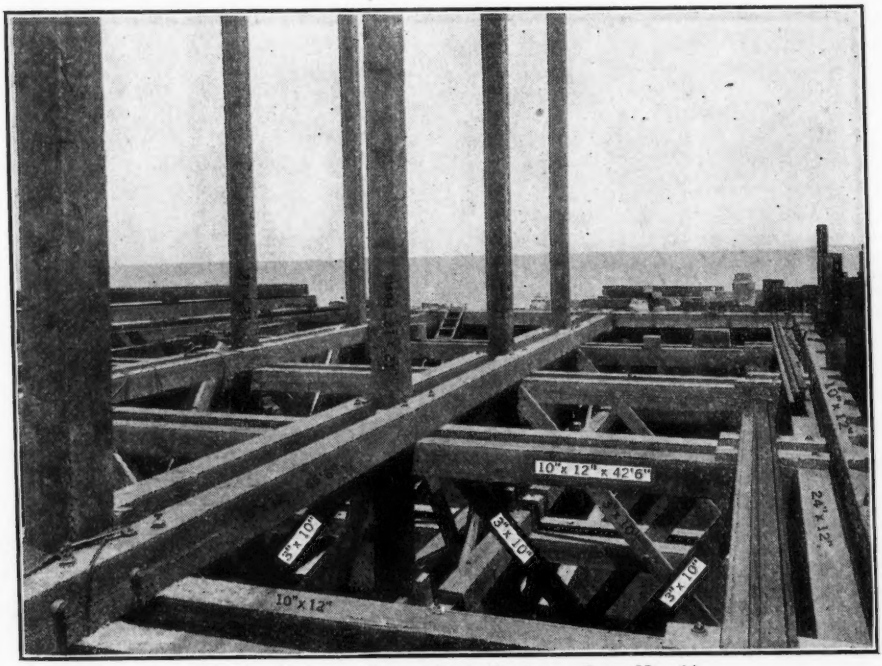


FIG. 6.—TYPICAL COFFER-DAM BRACING, PIER NO. 11.

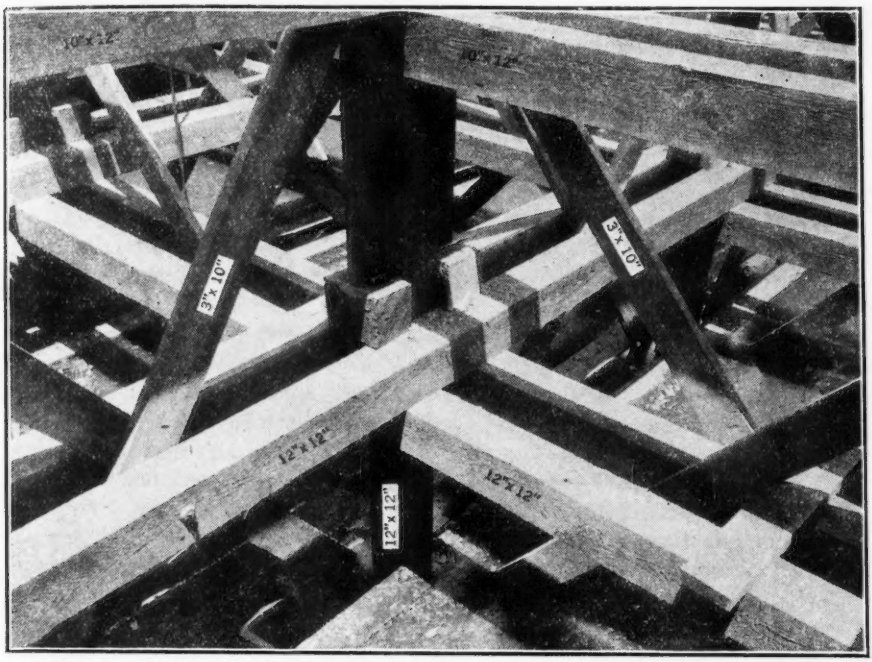


FIG. 7.—TYPICAL COFFER-DAM BRACING, PIER NO. 11, MARTINEZ-BENICIA BRIDGE.

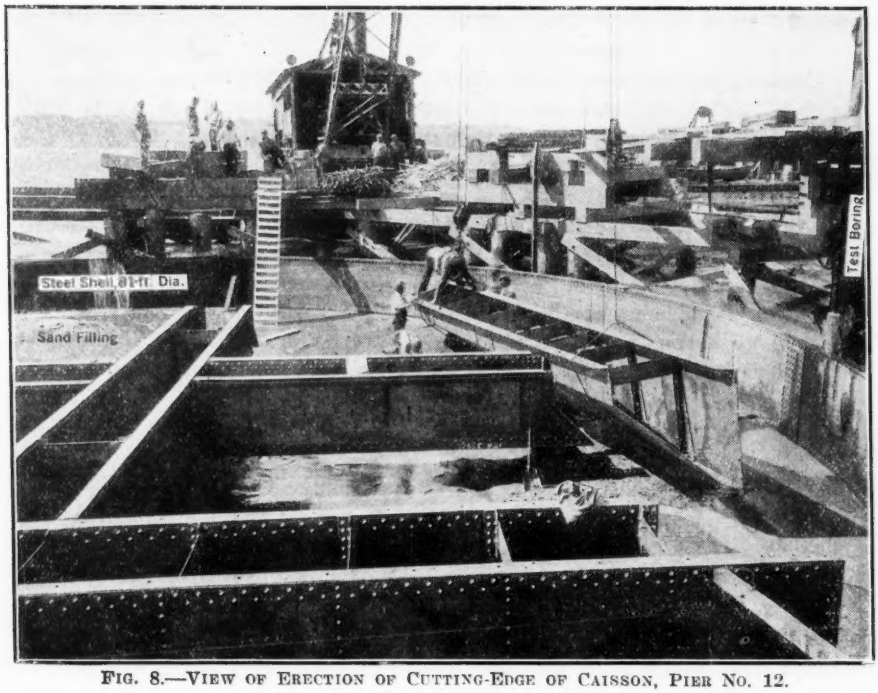


FIG. 8.—VIEW OF ERECTION OF CUTTING-EDGE OF CAISSON, PIER NO. 12.

winches and with cables passing over sheaves set at the top of the tower were erected. By this apparatus a steel shell that may be likened to a bottomless oil tank, 81 ft. in diameter, was erected and lowered through the water into the mud, which it penetrated to a depth of 15 to 30 ft., depending on the compactness of the material. This shell was built up of circular sections, 10 ft. in height, which were bolted together, permitting the later removal of all but the part in the mud.

The top steel shell was above high water and was filled with suitable sand dredged from up the river. The area of land thus created permitted the exact staking out and fabrication of the steel cutting-edge of the caisson that was then sunk to rock (see Fig. 8).

After the steel cutting-edge had been erected, the reinforcement steel was placed and the concrete poured in successive lifts of 10 ft. in height. All this work was done above water, which permitted the caisson to be constructed of reinforced concrete of the desired quality and strength. Exact centering and orientation was thus assured.

When the first 20 ft. of concrete shell was completed, the sand was excavated from the six dredging wells by clamshells operated from floating derrick barges. The caisson gradually settled into the sand deposit and the underlying strata, as the excavation from the dredging wells continued.

At the time the concrete caisson was started, four well holes were put down within the steel shell, by which the exact elevation of the bed-rock was determined at four points, conforming to the four sides of the caisson. This information determined the proper height of the caisson prior to sinking and also the elevation at which to begin construction of a timber coffer-dam that extended from Elevation — 20 to well above high water.

This coffer-dam (constructed of 12 by 12-in. timbers, drift-bolted, and properly caulked) was removable; temporarily, it rested upon, and was connected to, the concrete caisson by suitable tie-rods. The coffer-dam permitted pouring the distributing block (Fig. 3) and erecting the pier shaft from a depth 30 ft. below high water. It also effected a saving in concrete by the reduction of the large section of the caisson to the smaller section of the pier shaft, and created a temporary bridge seat or shelf which was used to support the steel bents of a falsework bridge span.

When the caisson was landed at or near bed-rock, and after the greater part of the material immediately above bed-rock was removed by hydraulic jetting and clamming, a deep-sea diver explored the bottom completely, telephoning the conditions existing within and under the caisson. He particularly noted the relation of the cutting-edge to bed-rock, and the amount of excavation still to be done.

In all cases considerable clay or gravel was found compacted under the partition walls and banked up against the cutting-edge. By means of the information thus furnished and by successive examinations the contractor was enabled to direct his jetting operations until the foundation area was reported satisfactory and in a clean condition.

The diver was able to supply accurate information as to the penetration of the cutting-edge into the foundation rock and in each case he brought up

samples of the rock encountered. This was perhaps the first time a deep-sea diver was used at such great depth of water in open caisson work. It enabled the railroad engineers to verify the diamond drill borings, to know definitely the nature of the bed-rock formation, and to be assured that the bridge piers were actually resting directly upon the rock. Work was continued until a full bearing was secured and an interlocked connection established between the concrete steel cutting-edge and bed-rock.

Thereafter, the pouring of the concrete seal, consisting of tremie concrete, began. This operation consisted of lowering a tremie pipe, 12 in. in diameter, sometimes to a depth of 150 ft. The upper part consisted of a funnel-shaped hopper and the entire pipe was supported by a line leading to a hoisting engine, so that it could be lowered and raised as desired. A plug of excelsior was placed in the tremie pipe and the pipe was filled with concrete, about 4 cu. yd. being required.

Thereafter, the column of concrete was not permitted to break and as it was deposited in the hopper the pipe was raised a few inches to permit the concrete to mushroom out at the bottom. The process continued without interruption until the dredging well was sealed to the desired height, and, successively, the six dredging wells were thus filled. After a sufficient quantity of concrete was in place the lower end of the tremie pipe remained from 6 to 10 ft. below the surface of the concrete during the pour.

At the beginning of the work, in order to study the condition of the tremie concrete, and to assist in visualizing the interior of the dredging wells, the tremie concrete was first brought up to an elevation 30 ft. above bed-rock. After the concrete had hardened the water was pumped from the caisson, the sides were washed down, and the mud and laitance were removed, this deposit being usually about $1\frac{1}{2}$ ft. thick. Samples were then broken out of the tremie concrete and tests made. In all cases a most excellent quality of concrete was found. Subsequently, being assured of the results obtained, the engineers permitted the tremie concrete to be brought, at one pouring, to near the elevation of the distributing block.

Within the recessed walls at, and resting upon, the prepared surface of the top of the caisson and the tremie-filled dredging wells, the distributing block was poured next. This was a large mass of reinforced concrete, 10 ft. thick, 50 ft. long, and 33 ft. wide (see Fig. 3).

The pier was completed by pouring the shaft to bridge-seat elevation in 10-ft. lifts. The wooden forms for this purpose were held in position by being bolted to the concrete section that was immediately below the section being poured.

The removal of the octagonal falsework, the pulling of the piling, the removal of the steel shell, and the dismantling of the timber coffer-dam followed in sequence after the pier was well above water. Such salvaged material was used again as work began on the next pier to be constructed. Fig 3, showing Pier No. 13, is typical.

The construction methods proved highly successful. In all cases the piers landed on bed-rock with only slight variation from the true center and inclination from the vertical. For instance, the caisson of the deepest pier

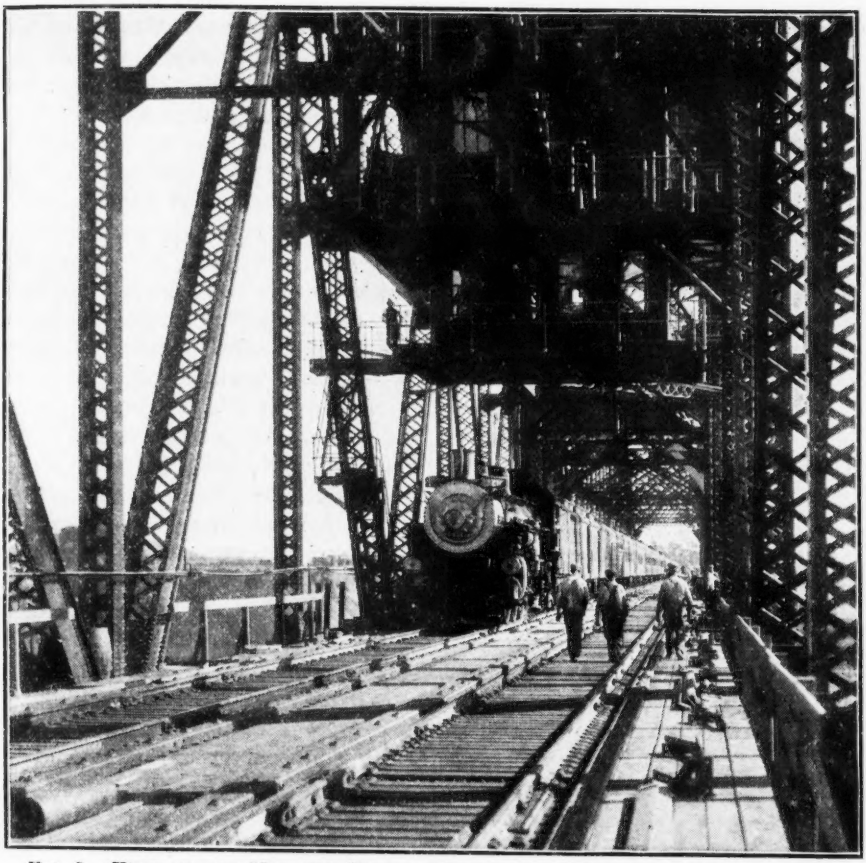

FIG. 9.—VIEW OF THE MARTINEZ-BENICIA BRIDGE, WITH TRAIN ON SPANS 12 AND 11.

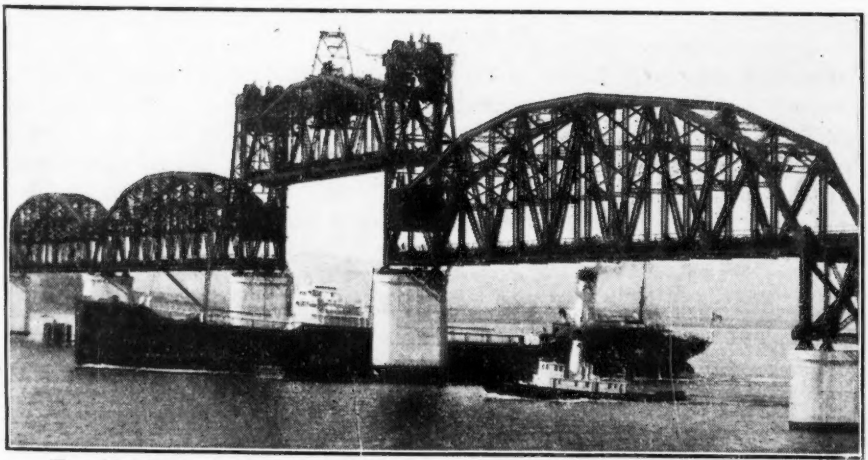

FIG. 10.—VIEW OF LIFT SPAN IN RAISED POSITION, MARTINEZ-BENICIA BRIDGE.

(No. 13) was off from true orientation by a southerly departure of 0.73 ft. and an easterly departure of 0.89 ft. The inclination from the vertical was 0.75 ft. in a height of 114.1 ft., which is equal to a batter of 150 on 1. Of course, even this slight error was not carried upward, as forms of the pier shaft were set to instrumental exactness.

Pier No. 12 may be used to illustrate the most accurate condition of placing. In this case the caisson departed from true orientation to the amount of 0.2 ft. northerly and 0.3 ft. westerly, the inclination from true vertical being only 0.14 ft. in 86.4 ft., which is equivalent to a batter of 600 on 1.

When one considers that these more or less unwieldy columns of concrete, with base areas of 2 400 sq. ft., and having a height in the one case equivalent to that of a 9-story office building, and weighing about 19 000 tons, have been set to the aforementioned accuracy, under the conditions found to prevail in connection with this work, the result appears to be marvelous.

To illustrate some of the construction difficulties the fact may be mentioned that at one time the caisson of Pier No. 12, was found to be out of level 1.3 ft. However, by confining the dredging to the wells on the high side, it was gradually brought back to a level position.

At another time when the steel shell was being set and sand was being placed at Pier No. 19, difficulty was encountered due to the tendency of the steel shell to settle to the north because the underlying strata was soft tule mud. The contractor righted the shell successfully, but due to the further settlement an additional 10-ft. ring of steel had to be applied. When next he attempted to fill the shell with sand, further settlement occurred on one side and the surrounding mud bulged upward.

Sometimes the caisson would fail to move downward, and as it was desired not to carry the dredging too far below the cutting-edge, small charges of dynamite were set off at the bottom of the dredging wells. These had the effect of causing the caisson to move. These instances illustrate the flexibility of the adopted method of sinking caissons.

## Superstructure

Fig. 9 is a view of the finished bridge, which is 5 603.5 ft. between abutments, accommodating two tracks spaced on 13-ft. centers. The elevation of the base of rail is 85.6 ft. above railroad datum, which provides a vertical clearance above mean higher high water of 70 ft. for the entire navigable distance of 4 050 ft. across the Bay. Provision also is made for a 328-ft. vertical lift span that will, when raised, clear mean higher high water by 135 ft. (see Fig. 10). The foregoing clearances are increased by 6.5 ft. when the tide is at mean lower low water.

The bridge is on tangent, except that spiraled 4° curves at each end extend outward on the viaduct sections. A length of 4 556.5 ft. of the bridge is on level grade, with approach grades of 0.45% on the north and 0.66% on the south end.

The superstructure consists of seven through truss spans of 526 ft. each, the distance center to center of piers being 531 ft., and also one vertical lift span,

328 ft. long; one deck span, 504 ft. long; one deck span, 264 ft. long; a viaduct approach on the south end of five 80-ft. girders and four 40-ft. tower girders; and on the north end a viaduct approach of one 100-ft. girder and three 40-ft. tower girders.

Truss-span lengths of 526 ft. are not necessarily the most economical, but navigation interests are better served by spans of 526 ft., primarily to permit the passage of long barge tows without the danger of side-swiping of piers. After conference with the engineers of the War Department and navigation companies, the 526-ft. length was agreed upon.

The towers for raising the lift span are integral with the spans adjacent to the lift span and rise to a vertical height of 139 ft. above the bridge seat. The viaduct towers and girders are of the usual standard design.

*Design.*—The superstructure was designed for a minimum live load on each track equivalent to Cooper's E-90 followed and preceded by a uniform load of 7 500 lb. per lin. ft. of track. The uniform load was used or omitted at either or both ends of the concentrated load to give maximum stress.

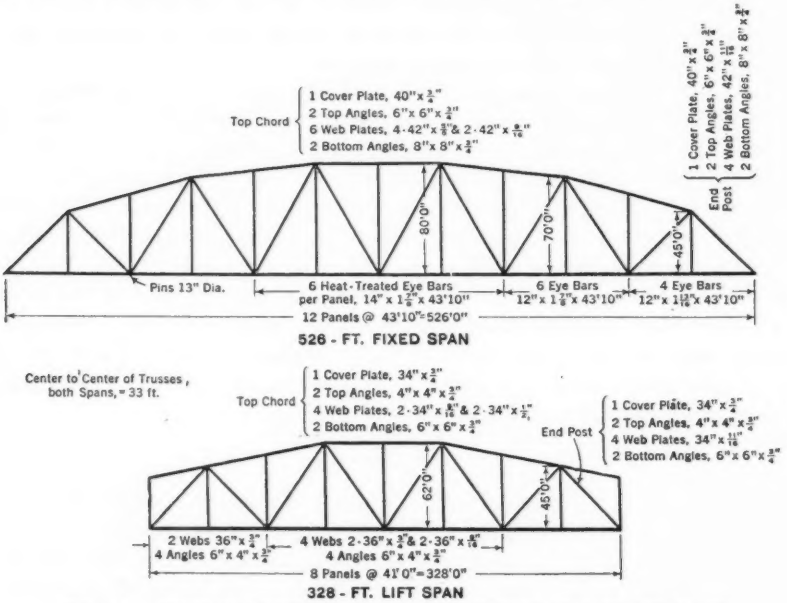
Top Chord
1 Cover Plate, 40" x 3/4"
2 Top Angles, 6" x 6" x 3/4"
6 Web Plates, 4-42" x 3/8" & 2-42" x 9/16"
2 Bottom Angles, 8" x 8" x 3/4"
1 Cover Plate, 40" x 3/4"
2 Top Angles, 6" x 6" x 3/4"
4 Web Plates, 42" x 11/16"
2 Bottom Angles, 8" x 8" x 3/4"
End Post
80'0"
70'0"
45'0"
Pins 13" Dia.
6 Heat-Treated Eye Bars per Panel, 14" x 1 3/4" x 43'10"
6 Eye Bars 12" x 1 3/4" x 43'10"
4 Eye Bars 12" x 1 13/16" x 43'10"
12 Panels @ 43'10" = 526'0"
526 - FT. FIXED SPAN
Center to Center of Trusses, both Spans, = 33 ft.
Top Chord
1 Cover Plate, 34" x 3/4"
2 Top Angles, 4" x 4" x 3/4"
4 Web Plates, 2-34" x 5/16" & 2-34" x 1/2"
2 Bottom Angles, 6" x 6" x 3/4"
End Post
1 Cover Plate, 34" x 3/4"
2 Top Angles, 4" x 4" x 3/4"
4 Web Plates, 34" x 11/16"
2 Bottom Angles, 6" x 6" x 3/4"
62'0"
45'0"
2 Webs 36" x 3/4"
4 Angles 6" x 4" x 3/4"
4 Webs 2-36" x 3/4" & 2-36" x 9/16"
4 Angles 6" x 4" x 3/4"
8 Panels @ 41'0" = 328'0"
328 - FT. LIFT SPAN

FIG. 11.—DIAGRAM SHOWING PRIMARY DIMENSIONS OF LIFT AND MAIN FIXED SPANS.

This loading is 50% heavier than the present standard and with it stresses one-third higher than standard are permitted. In calculating maximum stresses due to live load and centrifugal force in girders and trusses of spans carrying two tracks, both tracks were assumed to be loaded simultaneously and 90% of the specified live load was used.

Silicon steel was used for the principal members fabricated from shapes, and carbon steel for the remainder, except that the tension bars in the lower chord, consisting of eye-bars, were heat-treated. The total weight of the

FIG. 12.—FLOATING SPAN 19 FROM FALSEWORK TRESTLE, MARTINEZ-BENICIA BRIDGE.

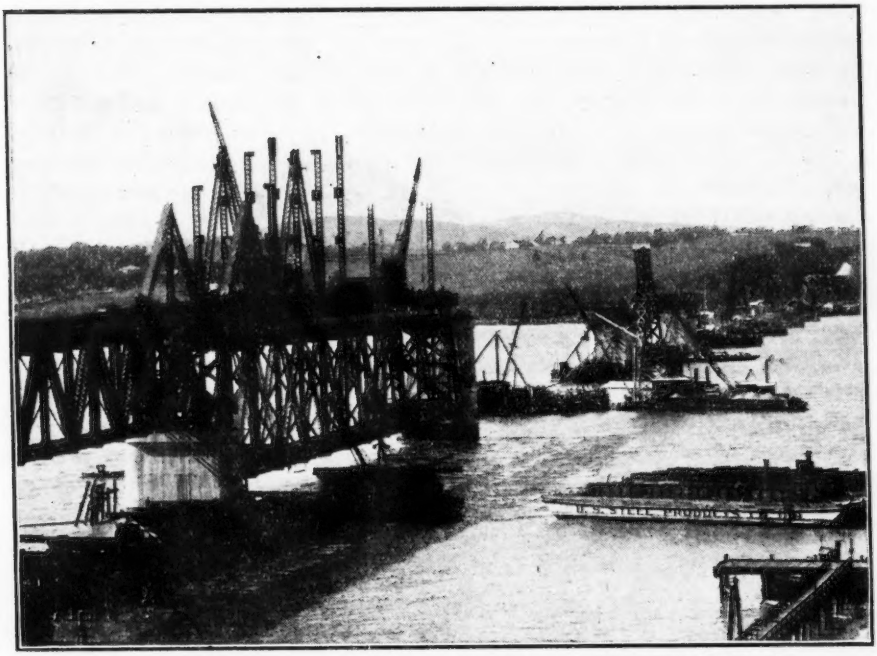

FIG. 13.—MARTINEZ-BENICIA BRIDGE: ERECTION OF SUPERSTRUCTURE AND CONSTRUCTION OF PIERS.

structural steel was 44 000 000 lb., comprising 25 000 000 lb. of silicon steel, 5 500 000 lb. of heat-treated eye-bars, and 13 500 000 lb. of carbon steel. The permissible unit stresses as used in the design were, as follows:

For axial tension in net section:

| | |
|---|---|
| Carbon steel | 21 300 lb. per sq. in. |
| Silicon steel | 32 000 lb. per sq. in. |
| Heat-treated eye-bars | 36 000 lb. per sq. in. |

For axial compression in gross section:

Carbon steel.... $20\,000 - 70\,\frac{l}{r}$ (up to 16 500 lb. per sq. in.)

Silicon steel.... $30\,000 - 105\,\frac{l}{r}$ (up to 24 750 lb. per sq. in.)

The primary dimensions of the superstructure are shown in Fig. 11. The lower chords consist of heat-treated eye-bars; all other truss members are of rolled structural shapes. The lift span, shown in Fig. 10, is a riveted truss throughout.

## Construction

*Steel Erection.*—Various methods of erection were considered, and it was deemed impossible to use timber piling successfully and economically owing to the depth of water, the softness and depths of the underlying mud, the swiftness of the current, and the height of the piers above water; nor was it deemed practicable to use the alternative method of "barging" the completed spans into position. An economical and satisfactory falsework method which avoided driving piling under the main spans, was adopted by the contractor. The steel was erected from the south or Martinez end, progressively working across the Bay to the Army Point end. The steel yard, therefore, was established at and near the south bridge-head, the railroad tracks, including the main-line tracks, being used for storage and switching purposes.

Proceeding from the abutment the tower posts and girders, comprising 560 ft. of viaduct, were erected by derrick car as far as Pier No. 10. Temporarily, the last pair of girders of the eastward track were used for a turnout approach to a falsework trestle that was driven immediately adjacent to and up stream above Pier No. 10 (see Fig. 12). This falsework trestle was provided for the erection of the falsework span, but the 264-ft. deck span was erected on it as a matter of convenience and was floated into its true position by means of barges and placed upon Piers Nos. 10 and 11. Next, Span 19, between Piers Nos. 19 and 20, was erected on the same falsework. This span, however, was used as falsework eight successive times in the erection of the remaining spans of the bridge. The procedure was as follows: After erection, Span 19 was floated from its falsework into position between Piers Nos. 11 and 12 (see Fig. 13) where it rested upon two temporary steel bents that extended to the shelf formed by the top of the caisson at Elevation — 20. These bents were secured to the pier shaft.

When the span had been set in place the elevation of its top chord was such that the necessary blocking to support the permanent span could be

placed. Permanent spans were then built as follows: (1) Lay the floor system and lower chord; (2) erect the vertical posts, diagonal web members, and top chord; (3) jack up the completed span to release the falsework truss; (4) float the falsework span into position between the next piers; and (5) lower the completed span on its piers. To erect the lift span, the falsework span was placed back on the falsework bents and cut apart at a panel to make it of proper length to go between the piers.

After Span 18 was completed, the falsework span was finally floated into position as a deck-truss span for connection with the north viaduct approach.

*Connecting Railroad Tracks.*—The finished project included the construction of the equivalent of 20 miles of single track, and the reconstruction of 24 miles of branch line to main-line track.

On the north end the eastbound track descends on a 2.2% grade to a connection with the main line in a distance of 0.93 mile. This steep grade is permissible due to the fact that the train movement is descending from the high elevation of the bridge to the lower level of the existing main line, and a flatter grade is unnecessary and uneconomical. The reverse is true of the westbound movement.

The operating requirement is that the locomotive load be not reduced on full-tonnage trains moving from the north or east toward the west over the bridge. Therefore, it was necessary to build 3.64 miles of track on an ascending grade of 0.45% to attain the bridge elevation. This required an entirely new location, leaving the main line at Goodyear Station and developing a supporting grade line on the foothills to the north (see Fig. 14). The line

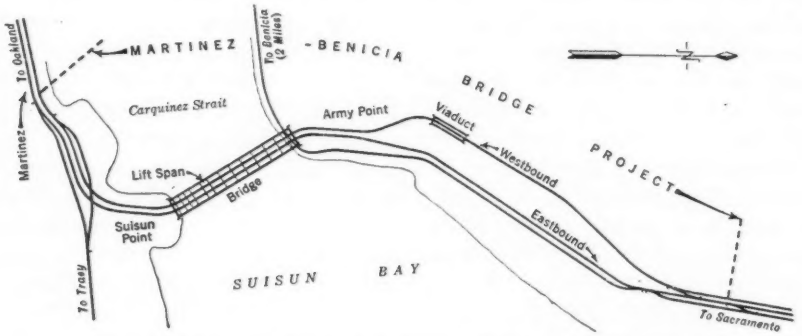


FIG. 14.—PLAN OF MARTINEZ-BENICIA BRIDGE AND APPROACHES.

crosses an arm of Suisun Bay, one of the inlets or marsh areas previously mentioned. The height of fill was 50 ft. and test wells proved the soil to be in the nature of peat, saturated with water, for a considerable depth. It was concluded, therefore, that it would be uneconomical to make an embankment, because the subsidence would be too great and the track unstable.

The engineers were not without previous knowledge of the action of such marsh areas, inasmuch as the present main lines in this territory, for a distance of 10 miles, have been maintained under difficulties for years. The subsidence has been as much as 80 ft., even with the subgrade near the natural

surface. Therefore, the choice was a steel viaduct, 1 860 ft. long. It consists of twenty-one 60-ft. girder spans and twenty 30-ft. girder tower spans, having concrete abutments and pedestal piers of concrete, each resting on 16 piles. The maximum length of piling was 80 ft. below the cut-off, which is below the water-table.

On the south end of the bridge the approach consists of a double track on a maximum 1% grade, extending in the direction of Oakland to Martinez Station. It was found impracticable, except at unjustifiable expense, to obtain an easier grade. However, 1% for eastbound traffic was not considered very objectionable as the grade would not restrict passenger trains unduly, and freights can be handled with "helper" engines, which will always be available, due to the large number of industries at Martinez requiring switching service.

Furthermore, most of the freight trains moving to and from San Francisco, and points to the south, will be handled over another approach, when the San Ramon Branch is reconstructed. This approach to the bridge will be on a 0.6% ascending grade eastward. Freight destined for Oakland (or the East Bay territory, as it is commonly called), and all passenger trains will still continue to move *via* Port Costa. A range of mountains extends in a north and south direction between the bridge and Oakland, making it necessary to follow the Bay shore on the north, or through Niles Cañon on the south, to reach either Oakland or San Francisco.

To facilitate such a movement, therefore, a second track, 4.4 miles long, has been constructed from the bridge to a connection with the existing double track at Port Costa. Part of this track is along the Suisun Bay shore and it has been riprapped as a safeguard against wave action.

The construction of the double-track aproach on the Martinez end involved some unusual problems. There, again, the engineers faced the problem of running track over marshland. The embankment was not as high as that on the north approach, however; the mud was shallower and below that there was hardpan. Other conditions also tended to make it desirable to construct a solid embankment. This embankment subsided during construction and the mud strata bulged outward, but excellent filling material was available economically and the job was accomplished without difficulty.

*Miscellaneous Structures.*—A concrete underpass was constructed through this embankment so that the westbound main line to Tracy could pass under the double track leading to the bridge. A 20-ft. vehicular subway was also constructed through the embankment.

Various other concrete structures, such as conduit boxes to handle numerous industrial oil pipes were constructed. Likewise, an overhead bridge was built to pass an industrial railroad over the main lines and sidings of the railroad (see Fig. 14). All structures were on pile foundations, driven into the mud strata to refusal, with cut-off below water level.

The main lines are equipped with automatic block signals and in addition to the signals and interlocking plants an automatic train-stop system is installed on each side of the lift span. The control of the bridge-operating machinery, including the locks, rail locks, and signals, are handled by an

electro-pneumatic interlocking machine with seven levers, provided with the customary mechanical approach and electric locking to insure that each function is operated in proper sequence and completely protected.

Each pier of the lift span is protected by a creosoted pile fender to permit the safe passage of sea-going vessels. This protection consists of a nose dolphin of fifty piles with two rows of pile spring lines leading to the piers. The spring lines are carried along the face of the pier by two rows of 12 by 12-in. timber fenders. The down-stream dolphin extends 46 ft. below the pier and the up-stream dolphin, 76 ft. above it. Protection is also provided at the other piers for smaller river craft.

All necessary navigation aids are provided and the top of the towers and the top chords of the bridge have flashing beacons for the benefit of air craft.

*Concrete Tests.*—Specifications for concrete required a strength of 2 500 lb. per sq. in. in compression and 400 lb. per sq. in. in tension within a 28-day period. After making exhaustive examinations of the available concrete aggregates, it was decided to require the cement to be fine enough that 85% would pass a 200-mesh sieve, and to use 3 lb. of diatomaceous silica per 100 lb. of cement, mixed with the cement at the cement plant. The concrete used in the piers, except the tremie concrete, was specified to be 1:2.5:5 mix by volume, or 1:2.4:4.6 by weight, with a water ratio of 0.9.

A well-graded crushed rock up to $2\frac{1}{2}$ in. in size, with sharp clean sand and 5 sacks of cement per yd. were used in this mix, and in the tremie concrete, $5\frac{1}{2}$ to $6\frac{1}{2}$ sacks per yd. were used, with $1\frac{1}{2}$-in. crushed gravel. The cement was placed in bins assigned to the particular job and after the tester approved the quality of the cement in it, the bin was sealed. Shipments to the job were made from this bin only.

Test cylinders were taken daily of the concrete poured. These were tested in the Railroad Company's laboratory for 7, 14, 28, 60 and 90-day curing periods. Greater strength was secured than was originally expected, as will be reflected by the following averages:

| Age of test cylinders, in days | Ultimate compressive strength, in pounds per square inch |
|---|---|
| 7 | 1 963 |
| 14 | 2 506 |
| 28 | 3 024 |
| 60 | 3 444 |
| 90 | 4 030 |

Because of the temperate climate, freezing and ice troubles were not encountered, but it was deemed necessary to consider the action of salt water on concrete. Therefore, that part of the concrete from a few feet below low water to a few feet above high water is formed of a rich 1:2:3 mix, using $1\frac{1}{2}$-in. rock aggregate. This concrete was poured dry and was subject to special spading and tamping so that a dense mixture was obtained. The surface was also treated to two coats of a coal-tar paint, selected after tests proved that the paint being used penetrated into the pores of the concrete.

*Layout Work.*—Ruling lines were established and caissons and piers were spotted by means of triangulation (see Fig. 15). First, a primary base line, 4 054 ft. long, was established and measured by an Invar tape. From this a secondary base line was projected in a location near the south abutment, the triangulation stations of which were intervisible with other primary stations

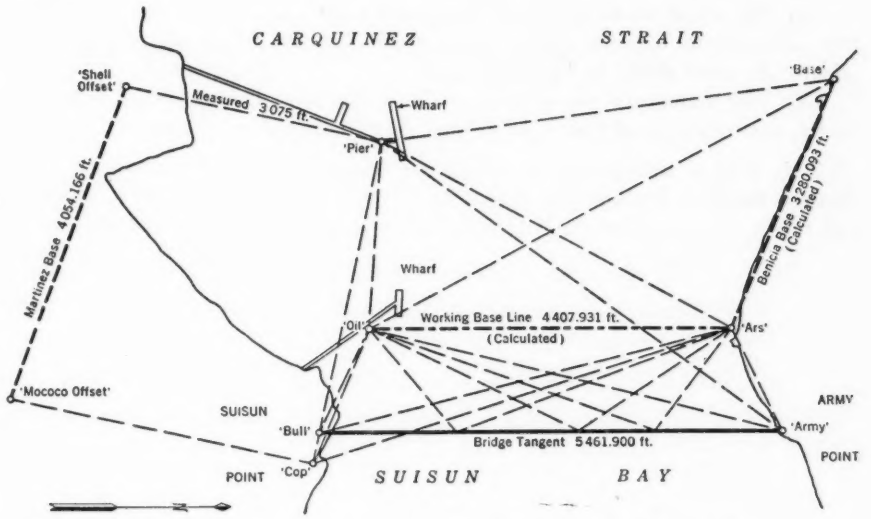


FIG. 15.—BASE-LINE TRIANGULATION, MARTINEZ-BENICIA BRIDGE.

established on each side of the Bay. Another base line was established on the north side. Four triangulation stations, two on each side of the Bay, were thus established, by which all operations in connection with the sinking of the caissons and erection of the piers were controlled with exact mathematical accuracy.

*Grading Operations.*—Extensive grading operations were required for the construction of the approaches. The contractor on the south end excavated and placed about 400 000 cu. yd. of material. The principal equipment consisted of two, 2-yd. steam shovels, operated by electricity, one 1-yd. drag-line, and one shovel operated by gasoline; and sixteen 10-yd. dump trucks, caterpillar tractors, and "bull-dozers." The contractor on the north end utilized dirt-moving equipment of ingenious design, his own development and manufacture. The principal equipment consisted of a 2-yd. Diesel steam shovel; 16-yd. heavy-duty dump carts and scrapers; and 60-h. p. caterpillar tractors. The dump carts were of all-steel welded construction; they were self-dumping and were mounted on caterpillar treads and hauled by caterpillar tractors. By means of this equipment about 500 000 cu. yd. of material was excavated.

## Conclusions

The following unusual features of design and construction are worthy of special notice:

(1) The intensive examination into earthquake fault conditions and the design of bridge piers to resist earthquake shock;

(2) The creation of artificial islands of sand within steel shells by which the all-concrete caisson bridge piers were formed above water and sunk within the steel shells to bed-rock; and

(3) The temporary use of one of the permanent deck spans as falsework in the construction of the other bridge spans across deep water.

This bridge was placed in operation successfully on October 15, 1930. During the first 24 hours of operation the lift span was opened four times, and thirty-nine passenger and freight trains were handled over the bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

# PRE-QUALIFICATION OF CONTRACTORS[1]

By C. J. Tilden,[2] M. Am. Soc. C. E.

Synopsis

A definition of pre-qualification of contractors is offered for discussion in this paper. This is supported by a brief historical comment, a statement as to legal aspects, and opposing viewpoints. Advantages and disadvantages are presented concisely, and the conclusion is reached that some form of pre-qualification is desirable from every point of view. Conditions must not be made so rigid, however, that new contracting firms would be prevented from entering the field.

## Definition of Pre-Qualification

The principle of pre-qualification may be defined as "the determination of responsibility before plans and proposals are issued for bidding." This determination is based on: (1) The contractor's previous experience; (2) the kind and condition of his equipment; (3) his record on previous work of a similar character; (4) his financial condition; and (5) his character and general reputation. A more extended definition was incorporated in the California Law of 1929 (Chapter 644):

"The Department of Public Works may, within its discretion before furnishing any persons proposing to bid on duly advertised public work, with plans and specifications for the proposed public work, require from any such person answers to questions contained in a standard form of questionnaire and financial statement, including a complete statement of the person's financial ability and experience in performing public work.

"Whenever the Department of Public Works is not satisfied with the sufficiency of the answers contained in such questionnaire and financial statement if may refuse to furnish such person with plans and specifications on any such duly advertised public work, and the bid of any person to whom plans and specifications have not been issued must be disregarded."

Note.—Discussion on this paper will be closed in December, 1932, *Proceedings*.

[1] Presented at the meeting of the Highway Division, New York, N. Y., January 22, 1931.

[2] Strathcona Prof. of Eng. Mechanics, Yale Univ., New Haven, Conn.

In a bulletin issued by the Bureau of Municipal Research, Philadelphia, Pa., in April, 1929, the following comparison was given:

"Prequalification is but an extension of the principle applied to the professions, where the possession of theory and practice must be demonstrated by the applicant and attested by diploma and State board; and in the trades, where those who would be masters must serve apprenticeships."

It is seen from the foregoing that the principle of pre-qualification is not new. It has been used to advantage for many years in private work, where the owner receives bids only from a restricted and especially invited list of contractors. The attempt to incorporate into law the principles defined herein is comparatively new, however, and has received its chief stimulus from the rapid expansion of highway building in the last few years.

## History of Pre-Qualification

It has always been a generally accepted principle that municipal and other authorities had the right to withhold the bidder's sheet from a contractor to whom the award would not be made, no matter how low his bid. Similarly, public officials have always had the right to require some kind of statement of the prospective bidder's qualifications. This is usually in the form of an answer to a more or less carefully prepared questionnaire. It was incorporated into the State Laws of Wisconsin in 1925, Iowa in 1927, and California in 1929. It is required or enforced either under existing specific statute or by administrative procedure in a number of other States, and, in addition, it is used in many municipalities. Proposed legislation on the subject failed, however, in Oregon in 1929, the chief cause being a provision in the bill to the effect that if a prospective bidder takes an appeal (from disqualification), no bids shall be opened or made public until this appeal has been determined. Such a provision might evidently cause serious delay. In 1929, also, the Governor of Pennsylvania vetoed proposed pre-qualification legislation on the ground that it placed too much discretionary power in one city official, without opportunity for redress on the part of the disqualified bidder.

The recent movement in favor of pre-qualification seems to have been started about 1924 by the contractors themselves, at a meeting of the Committee of Associated General Contractors of America and the American Association of State Highway Officials. It has had the enthusiastic support of the better class of contractors ever since that time.

## Legal Aspects of Pre-Qualification

Apparently, the Courts have not yet threshed out thoroughly the legal principle of pre-qualification. A city ordinance providing for pre-qualification of bidders on municipal work in Philadelphia was the basis of a suit brought by a bidder. The Court decided that the ordinance was illegal, but on appeal the judgment of the Lower Court was set aside. The case was reviewed by the Supreme Court, which declared the ordinance unconstitutional, but made several suggestions as to changes that might be made, which would keep the intent of the statute and yet be within the Constitution. One

of these was the desirability of a Board of Review, composed of persons in no way connected with the department that was advertising for bids, in order to assure the bidders unbiased consideration.

This is naturally not satisfactory to city engineers. It means that they must give up a duty and responsibility that they are loathe to relinquish. Furthermore, it does not appear to be in the public interest, for engineers within the department are certainly the best fitted to handle that phase of the work. A new pre-qualification ordinance was passed by the City Council of Philadelphia and became effective September 5, 1930. This was written to include the provisions held by the Supreme Court to be essential.

## The Contractor's Standpoint

From the standpoint of the general contractor, pre-qualification is decidedly advantageous. Responsible firms generally are hoping that the public will differentiate between classes of contractors, giving due weight to reliability, financial responsibility, and character, rather than making cheap price the controlling factor. Frequently, only two qualifications are required, the first being the lowest bid, and the second a satisfactory surety bond. This often militates against the responsible contractor who enters into his agreement expecting to abide by the plans and specifications and who has had experience in the technique of his work.

The surety bond is by no means complete insurance of satisfactory performance. Pre-qualification, as defined herein, will benefit both the bidder and the public. The Associated General Contractors of America has expressed itself as strongly in favor of the principle and has established a Bureau of Contract Information to co-operate with public officials and others who are investigating performance records, reputation, and general responsibility of contractors throughout the country.

## Failure of Contractors

The public generally loses thousands of dollars annually in failures on contracts that have been let on the basis of the lowest bid to incompetent contractors. A large surety company has classified the causes of its losses, as follows:

| | Per cent. |
|---|---|
| Inadequate financial ability | 50 |
| Incompetency | 25 |
| Dishonesty | 15 |
| Miscellaneous items, such as weather, bad luck, etc. | 10 |

A study undertaken by the Bureau of Public Roads, United States Department of Agriculture, to determine the reason for unsatisfactory progress on Federal Aid highway projects finds poor management the most important factor (52.7% of the cases studied), with weather (26.4%) second on the list. Inadequate financial ability, construction difficulties, and minor causes are also mentioned.

### Disadvantages of Pre-Qualification

A number of objections to the principle of pre-qualification have been stated. It is claimed that competition is unduly restricted by the limitation put on the number of bidders and especially that small firms struggling to get a satisfactory hold in the industry are barred from bidding. It is claimed further that the usual requirement of a certified check is in itself sufficient evidence of competency. Objection has been made to the questionnaire that must be answered as a preliminary to putting the contractor on the preferred list, as an unwarranted intrusion into private affairs.

Public officials who are charged with the duty and responsibility of letting the work are naturally biased in favor of more competent firms of whose previous work they know. This, of course, is a serious disadvantage to the new or little known bidder. If the principle of pre-qualification is too rigidly followed, it will prevent new men from coming into the field, and as existing firms drop out, there will be no others to fill their places.

### Advantages of Pre-Qualification

One of the most desirable results of pre-qualification, from the business standpoint, seems to be the increased confidence felt by manufacturers of contractors' equipment, bankers, and the surety companies. From the engineer's point of view the advantages are obvious, because he can feel much greater assurance that the plans and specifications will be followed properly. The contractors have expressed the firm belief that the principle is of benefit both to the bidder and to the public, and have supported this belief with practical co-operative efforts which tend to simplify and facilitate putting the principle into practice where it has been established.

With sworn statements at hand as to the contractor's experience, equipment, finances, and the number of contracts he has at the time, engineers can feel more confidence in the prompt and satisfactory execution of the work, and the banks and surety companies can feel greater confidence in the financial and economic phases of the question. As a protective measure to the legitimate and responsible contractor it is an important advantage.

### Conclusions

There seems to be little doubt or controversy among intelligent and responsible individuals, either engineers or contractors, as to the wisdom of some form of pre-qualification. With this practical unanimity, however, there is the feeling that conditions must not be made too rigid or the statutes too exacting. Although it is highly desirable to protect the interests of the public and also those of the high-grade contractor, there should be enough leeway in the exercise of judgment of governmental officials to permit new contracting firms to enter the field.

Some form of questionnaire that will exhibit fully and clearly all the facts with regard to the bidder's financial responsibility, general character, and probable ability to complete the work satisfactorily, is highly desirable.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# FINANCING STREET AND HIGHWAY IMPROVEMENTS

## Discussion

BY R. W. CRUM, M. AM. SOC. C. E.

R. W. CRUM,[12] M. AM. SOC. C. E. (by letter).[12a]—By common consent some of the factors of the problems of highway financing are becoming settled. For instance, the principle that the users should pay directly the entire cost of the main roads (except that part borne by the Federal Government) has been widely adopted, and the States generally finance their primary road improvements and maintenance by means of license fees on vehicles and taxes on gasoline.

Of course, a part of the total sum raised by these imposts is derived from travel on roads and streets not included in the self-supporting system. However, the determination of that part of the total sum raised that is derived from travel on the main highways does not appear to be impossible. In a given case the data needed are the car-miles of traffic on the system in question and the total traffic on all roads and streets in the jurisdiction. Since the primary system is definitely located and limited in extent, traffic surveys will give that part of the needed data.

The total traffic can be estimated from the gasoline consumption, as revealed by the tax returns, or from the total number of trucks and passenger cars registered, together with the average annual mileage per vehicle, and valuable data for use in making such estimates have been compiled.[13] The ratio of primary road traffic to total traffic will also be the ratio of the tax derived from primary road use to the total tax, and can be utilized in making an equitable allocation of the funds.

This ratio, in connection with the annual cost of the primary road system, should indicate also the total money that can be equitably raised by taxes on

NOTE.—The paper by R. W. Crum, M. Am. Soc. C. E., was published in August 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1931, by Messrs. A. P. Greensfelder and C. R. Thomas; February, 1932, by J. C. Carpenter, M. Am. Soc. C. E.; April, 1932, by Samuel Eckels, M. Am. Soc. C. E.; and May, 1932, by Horace H. Sears, M. Am. Soc. C. E.

[12] Director, Highway Research Board, National Research Council, Washington, D. C.

[12a] Received by the Secretary July 18, 1932.

[13] "Automobile Operating Costs and Mileage Studies", by Robley Winfrey, *Bulletin 106*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa.

users of vehicles. As an illustration, consider some data from a certain State: The average daily traffic during 1929, at 360 stations, covering the 3 490 miles of the State primary highway system, was found to be 732 vehicles. The total average traffic per day on these roads was, therefore, 3 490 × 732, or 2 554 860 car-miles.

An estimate of the total traffic on all the roads and streets of the State was made from the returns of the gasoline tax. In 1929, this tax produced $6 794 000, which means that at the tax rate of 5 cents per gal., 135 880 000 gal. of gasoline were consumed during the year, or an average of 372 274 gal. per day. Assuming that an average vehicle in this State will go 14 miles on 1 gal., the total average car-miles per day were 5 211 836. Of these daily car-miles, the 2 554 860 on the State Highway System amounted to 49 per cent.

The total revenue from motor vehicle fees and the 5-cent gasoline tax in this year was $9 334 000, of which 49%, or $4 573 660, was derived from travel on the State Highway System. Allowing for reasonable increase in traffic over a 30-year period, the amount derivable from the traffic on this system, without change in tax rate, was estimated to be about $5 300 000 per year.

An estimate of the average annual cost over a 30-year period of improving and maintaining this road system was $5 239 000 exclusive of the Federal Aid to be expected during construction. The amount to be realized, therefore, was just about equal to the demand.

If the amount derivable from the traffic on the primary road system was insufficient to cover the annual road costs, the State would be faced with the necessity of deciding between the alternate policies of increasing the tax rates, or of using some money on the primary system that was generated by traffic on other roads and streets. The justice of the latter procedure would depend largely upon the need of the State as a whole for improved highway transportation facilities.

This discussion is offered merely as an illustration of an attempt at a rational analysis of one phase of the problem. In addition to the list of needed research projects in the paper, it is suggested that a detailed study of the various comprehensive traffic surveys that have been carried on in numerous States would probably shed considerable light on the fixing of rates of taxation on motor vehicle users and the equitable allocation of the funds raised by these taxes.

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Founded November 5, 1852

DISCUSSIONS

# CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

## Discussion

BY MORRIS MASON, JUN. AM. SOC. C. E.

MORRIS MASON,[14] JUN. AM. SOC. C. E. (by letter).[14a]—It is gratifying to note that there is much unanimity of opinion concerning the advisability of using more scientific methods of concrete control as exemplified by practice on the Kittitas Division of the Yakima Project.

Mr. Peck approves the use of the pamphlet, "Instructions for Concrete Inspectors" and states that it must have been successful in teaching the essential features of the control of the water-cement ratio on the job to practical-minded men "to whom anything resembling theory is anathema." The use of the pamphlet was indeed a success. The education and experience of the inspectors on this work ranged from men of little education and many years of experience, to college professors, out for the summer, with very little experience, but well founded in theory, and to young college men who, in the beginning, were deficient in either or both respects. There were also a number of able engineers among the force of inspectors. The uniformity and good quality of the concrete produced under the supervision of these men speaks well for the method of inspection. The writer is of the opinion that the employment of inspectors of the grade of Assistant Engineers (Civil Service rating) or higher is amply repaid. The effect of employing better inspectors is generally noted in the loss of friction between owner and contractor. For the best work the contractor should respect the ability of the inspector. A chief inspector who might appear on the job at any time, either day or night, proved useful in keeping the inspectors always on the alert.

Mr. Peck questions the use of admixtures and just how much the admixture used contributed to the workable properties of the concrete. It is hoped that the discussion by Mr. Ruettgers has answered this question. The

NOTE.—The paper by Morris Mason, Jun. Am. Soc. C. E., was published in October, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1931, by John Sanford Peck, Assoc. M. Am. Soc. C. E.; January, 1932, by Messrs. Edward W. Bush and Orrin H. Pilkey; April, 1932, by Messrs. Clifford A. Betts, and George C. Imrie; and May, 1932, by Arthur Ruettgers, M. Am. Soc. C. E.

[14] Engr., Standard Oil Co. (Indiana), Wood River Refinery, Wood River, Ill.

[14a] Received by the Secretary June 25, 1932.

writer agrees with Mr. Peck that, generally speaking, the qualities attributed to the use of admixtures, with the exception of workability, may be attained by the additional use of 5 to 10% of cement. For the most part, it was a question of the relative cost of cement and admixture in place in the concrete. Little unbiased information is to be had on this subject.

The analysis of "unbalanced bidding" which was given by Mr. Bush is concise and to the point. Almost any one with construction experience can cite examples in which the contractor has collected on the "cream" of the work and then has left further worries to the surety or the owner. A. P. Greensfelder, M. Am. Soc. C. E., has emphasized[15] the fact that unbalanced bidding should be guarded against. He states that one is likely to feel that practical experience warrants certain maneuvering of unit bid prices. According to Mr. Greensfelder there is usually enough gambling in construction without this, and it is only warranted through the viewpoint of equity of the owner when the engineers' quantities are themselves unbalanced. On many jobs the retained percentage is retained in name only. As Mr. Bush states during the first part of the work the owner may be holding his own money as the retained percentage instead of any sum that has been earned by the contractor. Where the contractor is responsible and will complete the work the total amount bid is the important factor. There are exceptional cases in which the work will be in progress a number of years and most of the large items will be completed within the first few years, leaving the miscellaneous low-profit items to be carried for some time. The contract for a large dam is of this type. The contractor may honestly be using the owner's money rather than other money on which interest must be paid for the larger part of his operating expenses. Mr. Bush believes that most engineers expect a certain amount of unbalancing unless they include in the set-up, items such as installation of equipment, organization expense, over-head, pumping, and coffer-dams. It is becoming general practice to include many of these in the list of contract items.

In most instances the experienced contractor will bid high on the small items that will possibly be increased in amount, and low on the large items that will be reduced in the final estimate. A contractor who has been operating in one particular line for some years may prove a better estimator than the engineer whose work may extend to many lines and locations. The writer agrees with Mr. Bush that the upper 50% of the bidders are of no importance in connection with a true cost analysis. As a general rule, the lower three or four bidders will be found to agree closely on the total bid.

Mr. Pilkey requests the writer's palliative for the faults of the competitive system of bidding. There is no easy solution of the problem; but more intelligent estimating, by only experienced engineers, will induce the submission of more balanced bids by responsible contractors. The safety features used in connection with the canals on the Kittitas Project was discussed by Mr. Pilkey. If a man were sucked into one of the siphons it is scarcely possible that he would reach the outlet alive. It is impossible even for a

[15] *Proceedings*, Am. Soc. C. E., February, 1932, p. 250.

deer to climb the sides of these canals, in which the slopes are 1 on 1¼. When the canals are empty the deer that accidently happen in the canal must be stunned and removed by hand. As was pointed out by Mr. Pilkey, some of the early canals that were not completed, would not have been abandoned if they had been under Government supervision. The U. S. Bureau of Reclamation has been able to complete work that would otherwise never have been attempted. Construction work on the storage dam at Lake Cle Elum (see Fig. 1) was begun August 18, 1931, and will be completed about October, 1933. This reservoir, with Lake Kachess and Lake Keechelus Reservoirs, stores water for the entire Yakima River Irrigation District. Little of this work could have been done under private supervision. The early canals were constructed when water was plentiful and when any measurements made were in terms of the miner's inch. As more land was put under irrigation it was necessary to prorate the water, and as more water was needed it was necessary to construct storage reservoirs.

The exact items used in Mr. Betts' analysis of savings due to effective concrete control are not given, but the writer feels that any concrete job of importance will show a comparable saving. The writer agrees with Mr. Betts that costly control is burdensome and that practical inspection, administered with common sense, can be made indispensable. With the batching equipment now on the market the control of batching quantities, including the allowance for surface moisture in the aggregate, is semi-automatic. The charts mentioned should be useful in a large plant where they might be made to a large scale and tacked to the wall. Mr. Betts quotes some strength tolerances in which no test specimens varied more than 15% and in which 90% of the specimens varied within 10% of the average. That is an additional point in favor of effective control. It is better control than could be expected in scattered small structures on the same job. The writer reported 78% on the specimens within 20% of the average of probably one hundred structures, which were poured under possibly forty different inspectors. Mr. Ruettgers' discussion presents a usable method of testing impermeability of concrete. The writer agrees with Mr. Betts that there is need of a standardized test for impermeability. Thus far, no one has proved a direct relation between strength and impermeability, although, in general, they are both improved by the same factors. The writer does not agree that, as a general rule, wetting the mix will increase the imperviousness, other factors remaining the same.

The writer is indebted to Mr. Imrie for the enumeration of changes made in the design of the Yakima River Pressure Tunnel during construction. This structure is a novel design of the type. It is interesting to note that of the 6 750 acres of land shown as held by the Northern Pacific Railroad Company, all but 639 acres had, up to February, 1932, been sold to *bona fide* settlers at the appraised value. A large part of this acreage was very rough terrain.

Unbiased testimony on the use of admixtures in concrete is scarce. The tests reported by Mr. Ruettgers are an important addition to the subject. The water-cement ratios given in the paper did not allow for the absorption

of mixing water by the admixture. The water-cement ratios reported by Mr. Ruettgers as 0.90 would become approximately 0.80 if allowance were made for 200% absorption by the 3% of admixture; this would possibly account for part of the 1% to 50% increase in strength due to the use of admixture. The method of testing for permeability, and the effect of admixtures on the imperviousness of concrete, as reported by Mr. Ruettgers, is also of interest. Engineers concerned with the design of concrete mixes for hydraulic structures should be much interested in his statement concerning this method and the tests upon which the statement is based. The tests to determine the weathering properties are also interesting and instructive. The evidence of men in charge of actual concrete placing generally substantiates the fact that the concrete containing admixture is much more easily placed; that is, it is much more workable, because it contains the necessary "oiliness" or "fatness" for ease in placing. This characteristic can not be obtained by the addition of extra cement.

The writer realizes the difficulty of presenting a discussion of a paper of this kind and wishes to express his appreciation to those who have contributed to it. In closing, he wishes again to express his appreciation to the Engineers of the U. S. Bureau of Reclamation who so kindly assisted in the preparation of the paper, in making suggestions, furnishing information, and correcting the manuscript.

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Founded November 5, 1852

DISCUSSIONS

# WIND-BRACING CONNECTION EFFICIENCY

## Discussion

BY MESSRS. W. C. HUNTINGTON AND PAUL NIELSEN, AND A. TORIGGINO AND ERLE L. COPE

W. C. HUNTINGTON,[57] M. AM. SOC. C. E., AND PAUL NIELSEN,[58] ESQ. (by letter).[58a]—The writers have been much interested in the field covered by Mr. Berg and in the valuable material which he has presented. During the school year, 1931-32, they have been conducting an experimental investigation of the type of connection shown in Fig. 4(*b*) of the paper. The types of specimens tested are illustrated in Table 7.

The riveted specimens were made by first cutting longitudinally along the center of the web a 240-lb. Carnegie beam of sufficient length to provide the desired number of specimens. The flanges of the two parts of the beam were then riveted together, as illustrated in Table 7, the arrangement of the rivets being such as to yield the desired specimens when the member was cut into 4-in. lengths. The rivets were driven by hand with a gun riveter and are equivalent to those obtained in field riveting. It is hoped that the investigation can be extended to include shop rivets.

Plugs are inserted in the ends of each rivet and bolt in such a manner that the deformation of the shank can be measured accurately. This arrangement is illustrated in Fig. 26. All deformation readings are taken with an extensometer, making use of an Ames dial. Each division on the dial corresponds to a deformation of 0.0002 in., and the fractional part of each division is estimated.

A comparison of the results secured from Specimens *A* and *C* (Table 7) indicates the effect of the outside row of rivets. The effect of the bearing between the flanges on the area outside the rivets is indicated by a comparison of Specimens *A* and *B*. The effect of initial tension on the stress distribution among the rivets is indicated by comparing the results of a riveted specimen

NOTE.—The paper by U. T. Berg, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: April, 1932, by Messrs. David Cushman Coyle, William R. Osgood, O. G. Julian, Harold S. Richmond, and Robins Fleming; May, 1932, by Messrs. L. E. Grinter, C. R. Young, C. H. Sandberg, N. A. Richards, Jacob Feld, Dana Young, and A. J. Wilcox; and August, 1932, by Messrs. R. D. Dalton and John J. Gould.

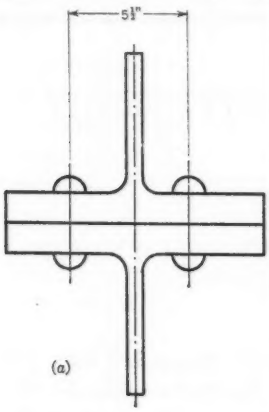
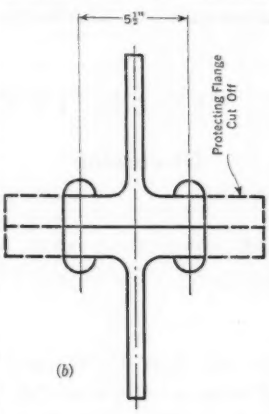
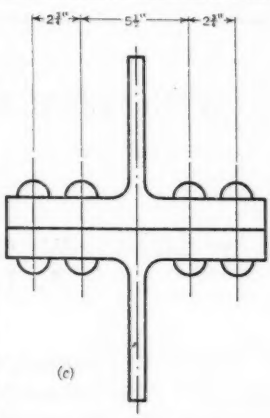
[57] Prof. of Civ. Eng., and Head of Dept., Univ. of Illinois, Urbana, Ill.

[58] Instructor in Gen. Eng. Drawing, Univ. of Illinois, Urbana, Ill.

[58a] Received by the Secretary April 27, 1932.

TABLE 7.—CONDITIONS OF TESTS ON CONNECTIONS

(Material: One 30-in., 240-lb. Carnegie beam, 4 in. long; $\frac{7}{8}$-in. rivets; and $\frac{15}{16}$-in. bolts.)

| | | |
|---|---|---|
|  |  |  |
| Specimen *A*: Rivets; initial tension | Specimen *B*: Rivets; initial tension | Specimen *C*: Rivets; initial tension |
| Specimen *F*: Bolts; initial tension | Specimen *H*: Bolts; initial tension | Specimen *E*: Bolts; initial tension |
| Specimen *G*: Bolts; no initial tension* | Specimen *I*: Bolts; no initial tension* | Specimen *D*: Bolts; no initial tension* |

* In order to hold these specimens firmly together it was necessary to have some initial tension in the bolts.

with those of a similar, bolted specimen in which the bolts are tightened only enough to hold the specimen firmly together. In Specimens *E*, *F*, and *H* the bolts are tightened to give the desired amount of initial tension as indi-

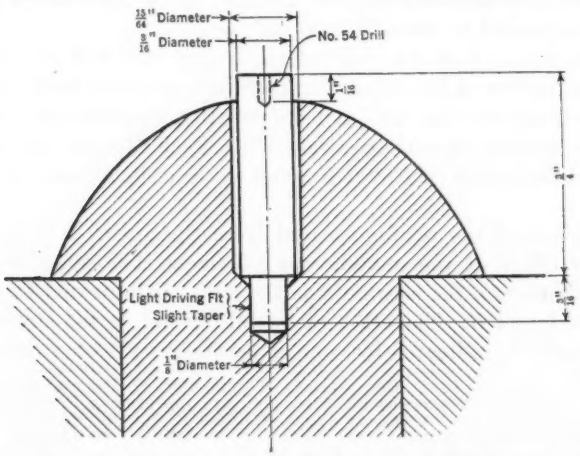



FIG. 26.—SECTION THROUGH RIVET HEAD SHOWING GAUGE-PIN.

cated by their deformation. These specimens give a quantitative measure of the effect of initial tension which can be varied as desired.

A part of the data secured in this investigation is shown on the load-strain diagrams in Fig. 27. The deformations given are in addition to the initial deformations due to the initial stress in the rivets or bolts. These diagrams will not be discussed in detail, but it may be readily noted that the outer rivets add materially to the strength of a connection of these

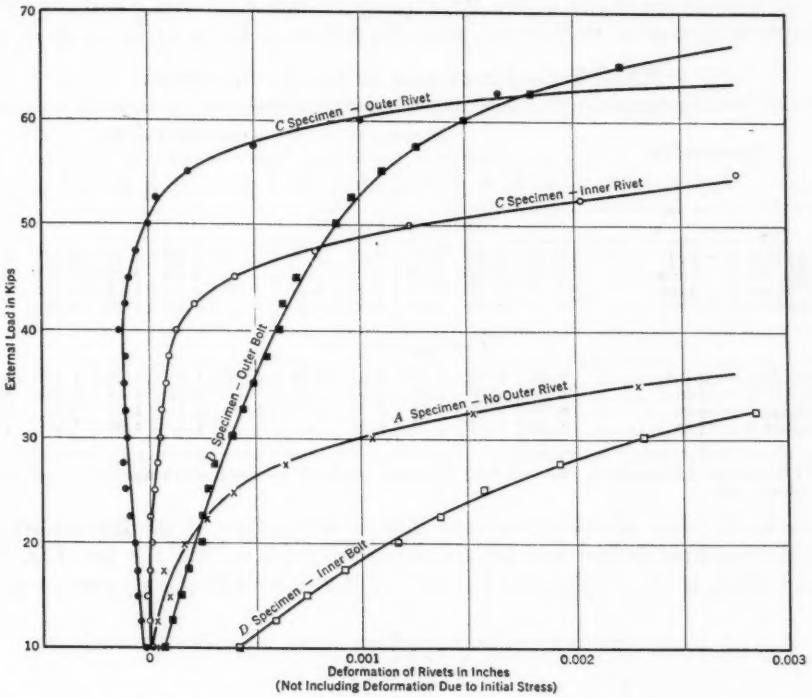



FIG. 27.—AVERAGE LOAD-STRAIN DIAGRAMS FOR RIVETS AND BOLTS.

dimensions. For instance, for the rivet stress corresponding to a deformation of 0.001 in. in excess of the initial deformation, the load carried by the *A* specimen that has no outer rivet is 30 000 lb., while the load carried by the *C* specimen with outer rivets is 49 000 lb. That the effectiveness of the *C* specimen is due to initial tension in the rivets is apparent when the load-strain diagrams for this specimen are compared with those for the *D* specimen in which the initial tension is only sufficient to hold the specimen firmly together. The effect of even this small initial tension is important, however, when calculations involving these diagrams are made. For instance, in some cases, the sum of the stresses in the rivets is less than the applied load, unless the initial tension is considered.

Another phase of the investigation indicates that the unit stress corresponding to the deformation of 0.001 in. used in the illustration in the previous paragraph, is below the elastic limit of the rivets and that the initial stresses in the rivets are about the same in the *A* and *C* specimens. The data

given in Fig. 27 are the averages obtained from three *A* and *C* specimens and two *D* specimens. These tests confirm other investigations which have indicated that rivets can be relied upon to carry tensile stresses.

A. TORIGGINO,[59] ESQ., and ERLE L. COPE,[60] M. AM. SOC. C. E. (by letter).[60a]—Some tests of full-sized wind connections, of cut I-beams, were made in connection with the design of the Mills Tower in San Francisco, Calif., in 1930. Fourteen specimens were tested, with rivets from $\frac{3}{4}$ in. to $1\frac{1}{4}$ in. in diameter,

TABLE 8.—DIMENSIONS OF GIRDER SPECIMENS

| Specimen No. | Dimensions, in Inches (see Key Fig. 28) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | *a* | *b* | *c* | *d* | *e* | *g* | *A* | *B* | *C* | *D* |
| **Bethlehem Sections** | | | | | | | | | | |
| Bethlehem *A* — 1¼-in. | 6.31 | 2.73 | 2.68 | 6.28 | 4.40 | 4.30 | 3.98 | 4.27 | 4.29 | 3.99 |
| Bethlehem *B* — 1¼-in. | 6.09 | 2.65 | 2.93 | 6.50 | .... | .... | 3.98 | 4.29 | 4.27 | 3.99 |
| Bethlehem *A* — 1½-in. | 6.17 | 2.57 | 2.72 | 6.32 | 4.50 | 4.50 | 3.99 | 4.28 | 4.27 | 3.96 |
| Bethlehem *B* — 1½-in. | 6.74 | 2.95 | 2.54 | 6.27 | .... | .... | 3.93 | 4.21 | 4.21 | 3.96 |
| **Carnegie Sections** | | | | | | | | | | |
| Carnegie *A* — 1¼-in* | 6.50 | 2.95 | 2.38 | 5.91 | 4.76 | 4.08 | 4.46 | 4.49 | 4.48 | 4.41 |
| Carnegie *B* — 1¼-in. | 6.96 | 2.28 | 2.74 | 6.28 | .... | .... | 4.53 | 4.58 | 4.56 | 4.54 |
| Carnegie *A* — 1½-in. | 6.40 | 3.80 | 2.85 | 6.67 | 4.57 | 4.68 | 4.52 | 4.51 | 4.51 | 4.52 |
| Carnegie *B* — 1½-in. | 6.45 | 3.06 | 2.61 | 6.18 | .... | .... | 4.52 | 4.52 | 4.53 | 4.60 |

* Dimensions of Carnegie *A*, 1¼-in. are with respect to center of beam web which was 0.50 in. off center of column web.

each having four rivets in tension. Detail dimensions of the heavier specimens, measured before testing, are given in Table 8, and the key Fig. 28, from which it is seen that the accuracy of the shop work was no greater than

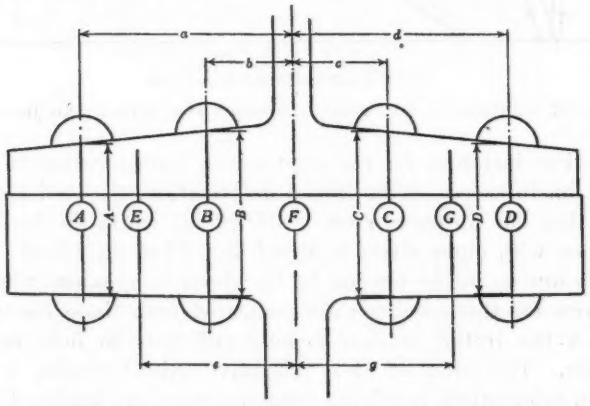



FIG. 28.—POSITIONS OF STRAIN GAUGES *A*, *B*, *C*, *D*, *E*, *F*, AND *G*.

that in ordinary building construction. All the tension rivets were driven with an ordinary pneumatic hammer.

[59] Cons. Engr., San Francisco, Calif.
[60] Cons. Engr., San Francisco, Calif.
[60a] Received by the Secretary May 24, 1932.

The jaws of the testing machine would not grip a width as great as the desired width of the specimen, and it was necessary to reinforce the webs of the 24-in. I-beam sections to insure them against failure. The webs of the girder-beam sections were forged to dimensions to suit the testing machine. The largest beam sections in use at the time the tests were outlined, were the 36-in. 300-lb. Carnegie beams and the 36-in. 260-lb. Bethlehem beams. Before the design of the Mills Tower was completed a 36-in., 300-lb. Bethlehem beam was produced, that has greater thickness of flange at the web than either of those tested, and this heavier section was used in the building. A 16-in., 384-lb., Bethlehem H-beam was used in all the specimens and showed no deformation under load.

Table 9 shows the loads that caused deformation of 0.005 in., 0.05 in., and 0.5 in. at the center. It also shows the ultimate strength in each case.

TABLE 9.—LOADS THAT CAUSE DEFORMATIONS OF 0.005 INCH, 0.05 INCH, AND 0.5 INCH AT THE CENTER

| Item | Specimen | Rivets | | Beam from which connection was cut | Loads, in Kips, Causing Deformation Shown at Center | | | Deformation at ultimate strength, in inches | Ultimate strength, in kips |
|---|---|---|---|---|---|---|---|---|---|
| | | No. | Diameter, in inches | | 0.005 in. | 0.05 in. | 0.5 in. | | |
| 1 | 1*A* — ¾-in.* | 4 | ¾ | 24-in. I; 100-lb. | 56.0 | 84 | ..... | 0.46 | 123.0 |
| 2 | 1*B* — ¾-in.* | 4 | ¾ | 24-in. I; 100-lb. | 60.0 | 85 | 120.0 | 0.48 | 120.0 |
| 3 | 1*A* — ⅞-in.* | 4 | ⅞ | 24-in. I; 100-lb. | ..... | 100 | 174.0 | 0.60 | 176.0 |
| 4 | 1*B* — ⅞-in.* | 4 | ⅞ | 24-in. I; 100-lb. | ..... | ..... | 180.0 | 0.60 | 181.5 |
| 5 | 1*A* — 1-in.* | 4 | 1 | 24-in. I; 100-lb. | 35.0 | 130 | 200.0 | 0.82 | 206.5 |
| 6 | 1*B* — 1-in.* | 4 | 1 | 24-in. I; 100-lb. | 62.5 | 133 | 207.6 | 0.65 | 212.0 |
| 7 | Bethlehem *A*—1⅛-in.* | 4 | 1⅛ | 36-in. B; 260-lb. | 63.7 | 125 | 173.0 | 1.46 | 209.0 |
| 8 | Bethlehem *B*—1⅛-in.† | 4 | 1⅛ | 36-in. B; 260-lb. | 102.0 | 121 | 195.0 | 0.80 | 205.0 |
| 9 | Bethlehem *A*—1¼-in.‡ | 4 | 1¼ | 36-in. B; 260-lb. | 53.5 | 157 | 225.0 | 0.90 | 253.4 |
| 10 | Bethlehem *B*—1¼-in.‡ | 4 | 1¼ | 36-in. B; 260-lb. | 126.0 | 140 | 220.0 | 0.95 | 251.0 |
| 11 | Carnegie *A*—1⅛-in.§.. | 4 | 1⅛ | 36-in. C; 300-lb. | 50.0 | 128 | 197.0 | 0.74 | 205.4 |
| 12 | Carnegie *B*—1⅛-in.‖.. | 4 | 1⅛ | 36-in. C; 300-lb. | 110.0 | 145 | 213.0 | 1.05 | 237.0 |
| 13 | Carnegie *B*—1¼-in.†.. | 4 | 1¼ | 36-in. C; 300-lb. | 76.0 | 154 | 258.0 | 0.94 | 293.6 |
| 14 | Carnegie *B*—1¼-in.†.. | 4 | 1¼ | 36-in. C; 300-lb. | 130.0 | 165 | 262.0 | 0.60 | 281.6 |

* Failed by "necking."
† Manufactured head snapped off.
‡ Manufactured head of Rivet *C* snapped off.
§ Rivet *B* failed in shear, in field-driven head.
‖ Rivet *B* failed by "necking."

Fig. 29 shows the typical make-up of the various sections. While the ultimate strength of the 1⅛-in. and 1¼-in. rivets (Items 7 to 14, Table 9) was satisfactory, their elongation varied greatly at failure. Most often the manufactured head snapped off to cause failure. The material in these rivets was not of uniform quality and the fractured surfaces showed a coarse crystalline structure. The tests show that rivets can be depended on to resist tension, and the writers believe that they are stronger in tension than in shear.

Test Specimen Bethlehem *A*, with 1⅛-in. rivets (Item 7, Table 9), is shown in Figs. 30 and 31.

Fig. 32 shows the deformation of a cut I-beam connection under a load of 175 000 lb. Failure occurred at 212 000 lb. The shape of the deformed connection is almost identical with that Fig. 3(*b*), of which the author

states: "It is readily seen that the I-beam stub connection (Fig. 3(*a*)), with a tension force of the value, 2 *S*, cannot deform as illustrated in Fig. 3(*b*)."

These tests show connections with more than one line of rivets (such as those shown in Fig. 4(*b*) and in Fig. 29(*c*)), to have considerable merit, and the writers cannot agree with the author's statement that, "connections with

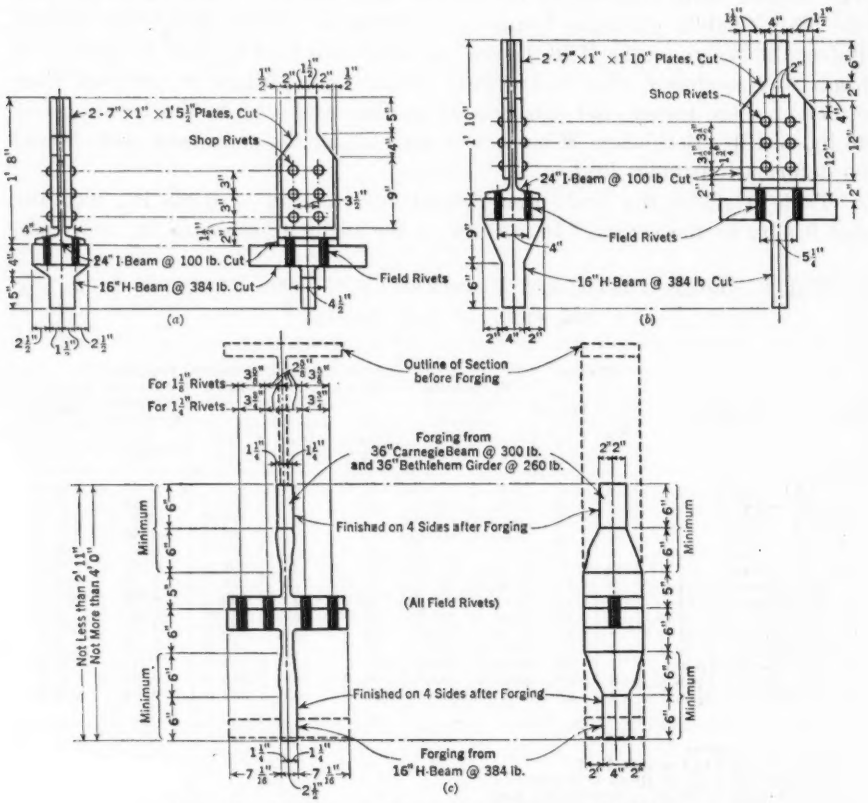

2 - 7"×1" ×1' 5½" Plates, Cut
Shop Rivets
24" I-Beam @ 100 lb. Cut
16" H-Beam @ 384 lb. Cut
Field Rivets
(a)
2 - 7"×1" ×1' 10" Plates, Cut
Shop Rivets
24" I-Beam @ 100 lb Cut
Field Rivets
16" H-Beam @ 384 lb Cut
(b)
Outline of Section before Forging
For 1⅛" Rivets
For 1¼" Rivets
Forging from 36" Carnegie Beam @ 300 lb. and 36" Bethlehem Girder @ 260 lb.
Finished on 4 Sides after Forging
(All Field Rivets)
Finished on 4 Sides after Forging
Forging from 16" H-Beam @ 384 lb.
Minimum
Not Less than 2' 11"
Not More than 4' 0"
(c)

FIG. 29.—MAKING OF TESTS SPECIMENS IN TABLE 9.

more than one line of rivets, such as shown in Fig. 4, have merit only because of increased shearing value."

The test record plotted in Fig. 33 shows that the elongation of the inside rivets, *B* and *C*, is approximately four times that of the outside rivets, *A* and *D*.

Many engineers are of the opinion that the architectural design of buildings should always permit the use of diagonal bracing or, at least, deep gussets or knee-braces. It should be remembered that although the structural parts of the building are of primary importance when safety is considered, they are only incidental to the use of the building. In structures of extraordinary height, the bracing assumes much greater relative importance and in such structures it may be proper to let the design of the bracing govern the architectural arrangement, but in most cases the utilitarian purpose of

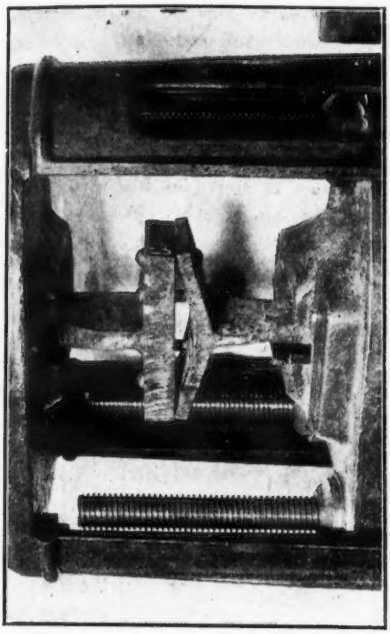


FIG. 31.—SPECIMEN BETHLEHEM *A*, 1⅛-INCH, AFTER FAILURE AT 209.8 KIPS.

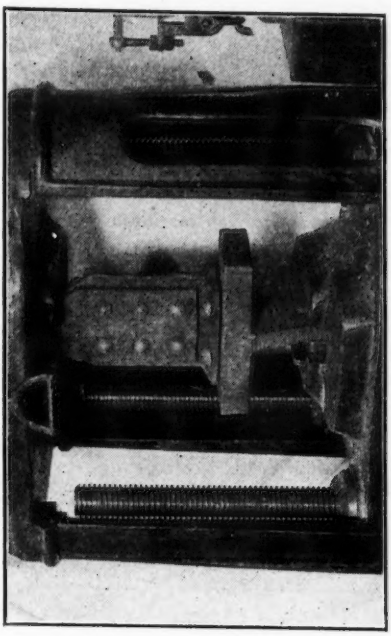


FIG. 32.—SPECIMEN 1 *B*, 1-INCH, AT LOAD OF 175 KIPS. FAILURE OCCURRED AT 212 KIPS.

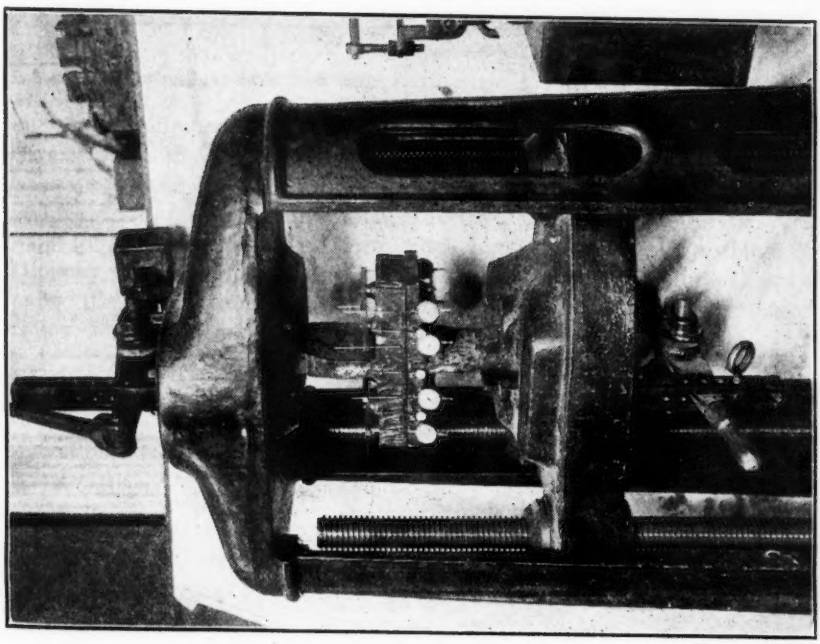


FIG. 30.—SPECIMEN, BETHLEHEM *A*, 1⅛-INCH, BEFORE LOADING, SHOWING DIAL DEFORMETERS ON RIVETS AND FLANGES.

the building should be the controlling factor. In other cases esthetic considerations do not permit deep connections or *X*-bracing.

The structure must have sufficient strength to resist the loads that will come upon it and it is the function of the engineer to provide this at the least cost and with the least interference with the requirements of the use of the building.

The shallow types of connections are not so rigid, are more costly, and are generally less efficient; however, they have some advantages over deep

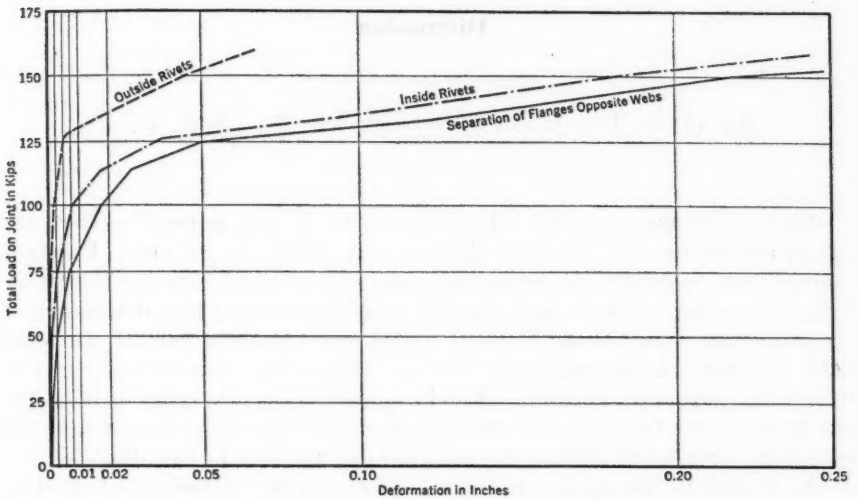



FIG. 33.—DEFORMATION OF SPECIMEN BETHLEHEM *A*, 1⅛-INCH RIVERTS UNDER LOAD.

gussets, knee-braces, and *X*-bracing besides their adaptability. One important advantage is the high ratio of ultimate strength of the connection to the working stress. In structures subject to forces of unknown magnitude, such as tornadoes and earthquakes, this is a very desirable characteristic. Frames with connections that will deform when highly overstressed, without failing (thereby relieving the bending stresses in the beams and columns) are much more likely to stand up than those with stiffer and more unyielding connections.

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DISCUSSIONS

# STEREO-TOPOGRAPHIC MAPPING

## Discussion

BY OTTO LEMBERGER, ASSOC. M. AM. SOC. C. E.

OTTO LEMBERGER,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—That surveyors and engineers in the United States have finally begun to use European methods and apparatus for stereo-topographic surveying purposes is particularly interesting to the writer. In confining his remarks to a description of the aerocartograph, Colonel Birdseye treats only a limited part of the broad field of stereo-photogrammetry, without giving the general theory upon which this apparatus is based. The introductory remarks relating to parallax which is the fundamental principle of stereoscopic measurements, should have been extended to include the fundamental equation of parallax.

As applied to this subject parallax is expressed as a difference of abscissa. The writer has attempted to clarify this point previously.[27] The principles may be demonstrated as follows: Look through two tubes at a pair of sharply pointed pencils, or one finger of each hand, held at the opposite ends. Only one pencil, or one finger, will be noticeable. Next move one of the objects across the end of its tube toward or away from the other object, and the impression is created to the eye that the motion is along the axis of the tubes rather than transverse. In stereoscopic surveying the term, parallax, indicates two conditions. In terrestrial surveying it indicates horizontal distance, as explained by the foregoing experiment, and in aerial surveying it indicates vertical height.

In general, only three types of stereoscopic plotting instruments have thus far proved to be sound, either as to theory or construction, for use in constructing maps on a scale of 1:5 000 to 1:30 000. These are the aerocartograph described by Colonel Birdseye (Fig. 11), the stereoplanigraph (Fig. 6), and the autograph (Fig. 7). These three are known in Europe

NOTE.—The paper by C. H. Birdseye, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930, and published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Theron M. Ripley, O. S. Reading, and Lowell O. Stewart; May, 1932, by Messrs. F. H. Peters, W. H. Crosson, Dwight F. Johns, and Douglas H. Nelles; and August, 1932, by Messrs. R. E. Ballester, and T. P. Pendleton.

[26] Vienna, Austria.

[26a] Received by the Secretary April 25, 1932.

[27] *Die Wasserwirtschaft*, 1919.

as space autographs and their construction is based on the principles of Porro and Koppe as developed in the Bildmesstheodolit, known in English as the photo-goniometer. The photo-goniometer, shown in Fig. 16, is an apparatus that enables one to measure points on a photograph as one measures them with a theodolite in the field. The same lens must be used in the photo-goniometer as in the camera with which the photographs were taken, or at least the lens in the photo-goniometer must have the identical optical qualities and the same focal distance as the lens of the aerial camera. Only in this way can the algebraic sum of the errors of the aerial camera and of the photo-goniometer be made equal to zero.

With an instrument constructed on the foregoing principles, it is possible to measure the tilt of the optical axis of the aerial camera as well as its lateral inclination, which is called the "swing" of the horizontal line of the photograph. When the tilt and the swing of an aerial photograph are known, measurements can be made directly on the photograph the same as with a theodolite on the ground. The idea of the space autograph originated with Dr. C. Pulfrich, but it was Gustav Heyde, following the principles of Dr. Reinhard Hugershoff, who developed the first practical space autograph. He combined two photo-goniometers and named the apparatus the autocartograph (Fig. 8). The process of adjusting the plates used in the three space autographs was developed by Dr. Otto von Gruber. While plates as well as films may be used in the aerocartograph and the stereoplanigraph, only plates are used in the autograph. In the writer's opinion, the latest model of the stereoplanigraph developed in 1930 can be considered as the most generally adaptable instrument of the three. The writer has described them in detail elsewhere.[28]

In all these instruments the photographs accommodated show only a comparatively small area, generally not larger than 20 sq. km. (7 sq. miles). This is one of the principal arguments against their use in surveying larger tracts of land. The space autographs described in the foregoing comments, used in connection with corresponding aerial cameras, have not proved their economic superiority in preparing small-scale maps of large territories in which there is no triangulation network. Captain Hotine has criticized European space autographs for this reason. He based his judgment on the many investigations made by the Geographical Section of the British General Staff. Two instruments constructed in England have been described by Captain Hotine[29] and Lieut. T. S. A. Salt.[30] One of these, the photogrammetric plotter, was said to be specially adapted to surveying larger areas, but apparently it has not achieved any considerable popularity. The instrument described by Lieut. Salt is the stereo-goniometer, perfected by H. G. Fourcade. This instrument is used in combination with "image triangulation," a process explained subsequently in this discussion, to obtain a contour map. Photographs for use in this instrument must have scarcely no tilt and must

[28] "La Fotogrametria Terrestre y Aerea. Su applicacion en la formacion de planos tecnicos y cartas geograficas," Sociedad Cientifica "Antonio Alzate," City of Mexico, Vol. 50, 1928.

[29] *The Geographical Journal*, February, 1930.

[30] *Internationales Archiv für Photogrammetrie*, VII Band, 1930.

be taken in a strip with an overlap of 60 per cent. With the stereo-goniometer, it is possible to produce a map on a scale of 1:20 000 at the rate of 4 sq. miles per hour. This progress requires a computer, a force of five draftsmen, and ground control equivalent to that of an ordinary secondary triangulation network.

In the stereo-goniometer overlapping photographs placed in two cameras are observed through a fixed binocular telescope system fitted with floating marks. The line connecting the two cameras may be called the air-base line; that is, the line joining the positions of the camera at the moments of successive exposure. This base line is called the polar axis. All movements of the two cameras as well as of two movable mirrors placed between the cameras and the telescope are referred to this axis. The axes of the mirrors coincide with that of the telescope system. One characteristic of this apparatus is that the devices for moving both cameras are entirely independent of those that govern the observation and the plotting. This simplified the design of the instrument which is similar in theory to that previously accredited to Dr. Koppe in which two movable cameras are observed from a fixed observation system, as in the case of the autograph.

In the autograph, however, all movements are referred to a horizontal plane in such a manner that the air-base line is considered as projected on the three space planes. In the stereo-goniometer the error of projection of a point on the photograph of each camera is corrected by means of mirrors that can be moved and rotated, whereas, in the autograph, corrections are made automatically by turning the plates around their principal points, and in their planes by a small swing angle. This error of projection in the autograph is due to the fact that the guide-rod that coincides with the camera axis, includes with the sighting axis of the fixed observation telescope other angles than the light ray includes with the axis of the air camera. The use of mirrors in the stereo-goniometer to effect perfect correspondence (parallelism to eye-base) of two images of a point when viewed in a stereoscope was first introduced in 1924 in the stereoplanigraph.

The stereo-goniometer has only a limited field of application. It requires photographs taken vertically, the maximum tilt being 2 degrees. It is practicable for producing maps on the photo-scale only; for example, 1:20 000, whereas in extensive territorial surveys a scale of 1:100 000 or less is desired.

Contrary to the opinions of Captain Hotine and Lieut. Salt, the writer considers that the plotting of contours and topography by means of plotting instruments equipped with hand-wheels and foot-operated disks is entirely feasible. He has instructed quite a number of topographical draftsmen (most of them lacking scientific education entirely) in the operation of automatic plotting machines, and the results have been excellent as long as the stereoscopic sight of the operator was good. Plotting with such an instrument and keeping it in good working order are necessarily two different functions, but the latter also is made possible by explicit instructions. Special radial triangulators should be used in "image" triangulation for good results. Their use reduces the number of draftsmen and makes it possible to do the work faster and better.

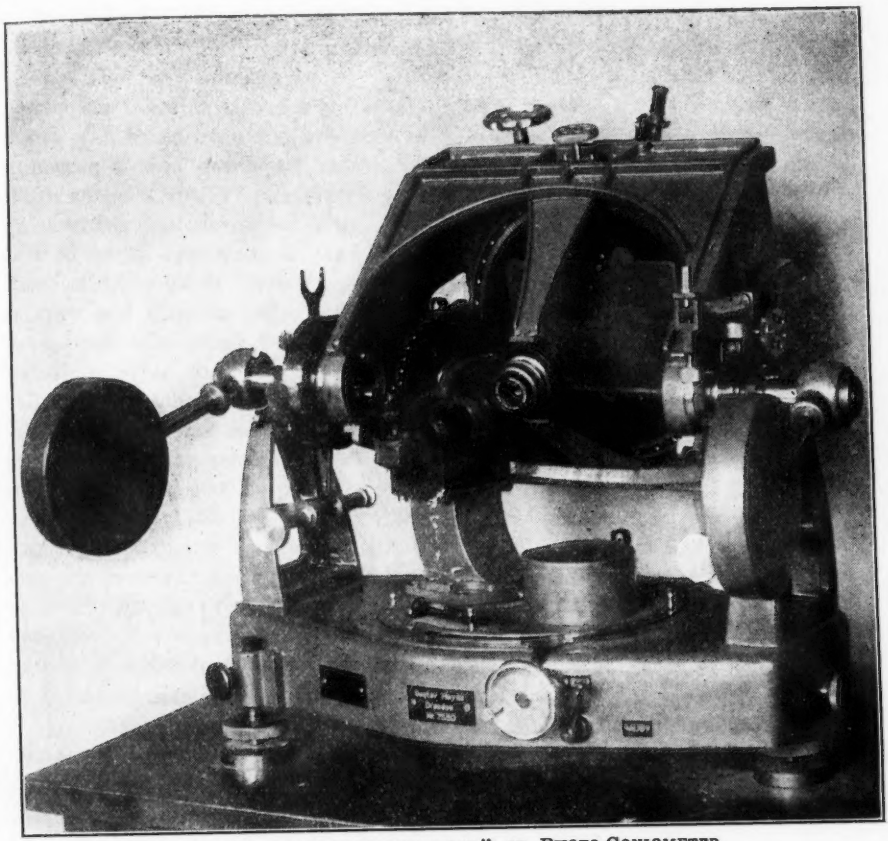

FIG. 16.—"BILDMESSTHEODOLIT," OR PHOTO-GONIOMETER.

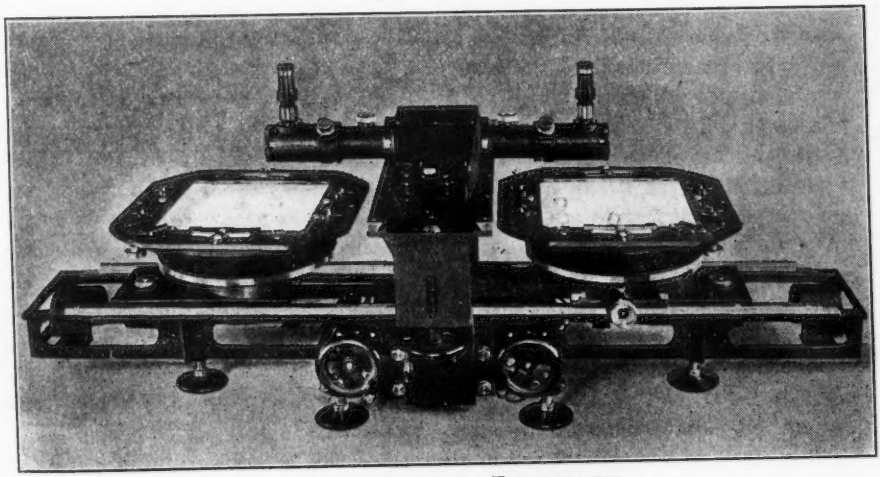

FIG. 17.—THE RADIAL TRIANGULATOR.

The recent development of a panoramic camera will probably make stereo-topographic mapping a paying proposition for small-scale mapping of large areas. The vast unsurveyed territories in America, Asia, and Africa can now be photographed to a scale as small as 1:100 000, and the maps can be reproduced to an even smaller scale if necessary. A new type of panoramic camera has been developed by Dr. Claus Aschenbrenner who is continuing the work of Capt. T. Scheimpflug. Long before the World War, Captain Scheimpflug had developed a multiple-lens camera composed of eight lenses as well as an apparatus to transform the seven oblique photographs to the plane of the central vertical one. At that time the art of flying was in its infancy and Captain Scheimpflug used free balloons. Now, that air routes have been established over so many large unsurveyed areas, the question naturally arose as to how these large areas could be surveyed economically. Dr. Aschenbrenner considered that there were two great obstacles to perfecting cameras for such use: First, the long focal distance of existing air cameras; and, second, their small image angles. The significance of the first obstacle is that the scale of the usual single-lens photograph is limited to 1:20 000 or 1:30 000, assuming 5 000 to 6 000 m. above ground as the ceiling, without using special oxygen equipment. According to Dr. Aschenbrenner an economical photo-scale for such surveys would range from 1:75 000 to 1:100 000, or even smaller. Such scales cannot be obtained everywhere, however. For example, the plateaus of Mexico and Colombia are at an elevation of 1 000 to 3 000 m. above sea level. In such cases an image scale of 1:60 000 may still be considered the limit.

Dr. Aschenbrenner developed a nine-lens camera with eight lenses grouped around the central one, all with focal lengths of 54 mm., or 2.1 in. The optical axes of these eight lenses are parallel to that of the central one, but prisms are placed in front of each of the eight lenses to gather light rays 54° from the vertical. By means of this disposition of lenses and prisms and by operating one shutter, the images from all nine lenses, covering a field of 140°, are photographed simultaneously on one 18 by 18-cm. section of film. Fig. 17 is a view of a radial triangulator used in determining control points as described subsequently. Fig. 18 shows the camera with the film magazine dismounted, and Fig. 19 shows the transformer used to make the assembled prints. Fig. 20 is an exact reproduction of the segments of an untransformed photograph taken to the scale of 1:60 000 on a film, 18 by 18 cm., from a height of 4 800 m. above the ground. Fig. 21 shows corresponding segments of the same photograph after transformation and assembly in a print, 27 by 27 cm., on a scale, of 1:90 000, covering an area of about 590 sq. km., or about 212 sq. miles. The particular advantages claimed for this camera over the American type of five-lens camera are the much shorter focal length, the use of only one shutter, and the fact that the transformed print has no blank gores between the wing segments.

By means of the instrument known as the radial triangulator the number of control points necessary in the panoramic process devised by Dr. Aschenbrenner is greatly reduced. The radial triangulator shown in Fig. 17 serves

to determine stereoscopically by means of polar co-ordinates the correct position of images of points with reference to the ground. After this has been accomplished a certain number of control points are determined stereoscopically without any ground measurements. This process has been called "image" triangulation by Dr. Aschenbrenner as all the angles are determined by observation of the images.

Specifically, the method is as follows: The two images of the same point on two overlapping photographs are rotated about their centers or principal points until they lie with their corresponding images on or parallel to the air-base line or the line connecting the two camera positions. If both images refer to the same flight strip, the air-base line corresponds with the line of flight. The principal points are assumed to replace the plumb-points (nadirs), which is permissible as long as tilts of the photographs do not exceed 3 degrees. To be sure that the objects at the center of one photograph are also included on the next photograph exposed, an overlap of more than 50% is necessary. The correct positions of both images are obtained when the adjacent photographs are properly oriented and separated so that corresponding images of the points are in true stereoscopic fusion. After this is once accomplished, lines of sight to corresponding points of photographic detail must intersect at single image points, thus producing a stereoscopic model of the landscape.

The next step is to determine the location of a certain number of control points, a process that was first devised and put into application by Captain Scheimpflug in 1898. The same method has been used by different men in various countries in succeeding years.[31] By connecting the principal points as stated, a network of triangles is drawn to which any point that appears clearly on different overlapping pictures can be referred. The scale of this image triangulation is known as soon as the distance between any two points, quite visible on the photographs, is known, as, for instance, between two-triangulation points. Independently, Dr. Aschenbrenner and Dr. J. Koppmair conducted mathematical investigations of the radial line method and proved that vertical photographs can be used as long as their tilt is not greater than 3 degrees.[32] In aerial topographic surveys some differences in the flying height are permitted as long as they do not cause greater errors of position than are consistent with the accuracy of the map itself. The positions of the control points as determined by image triangulation are correct and, therefore, independent of difference in the flying height.

Control points are determined, as follows: First, the pictures taken at each exposure of the nine-lens camera are transformed and printed on a single plate, 30 by 30 cm. Then, the principal points are marked permanently on the plate by a special device known as the cross straight-edge. The next step is to transfer the position of the principal point of one plate to its corresponding image position on the adjacent plate. For this purpose the plate-holders can be rotated and moved parallel to the line connecting the principal points. The two overlapping pictures supply the coincident images

[31] *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 792.

[32] "Apuntes sobre la rectificacion o transformacion de vistas aereas, tomades aproximadamente con ejes opticos verticales," *Ingenieria*, City of Mexico, June, 1930.

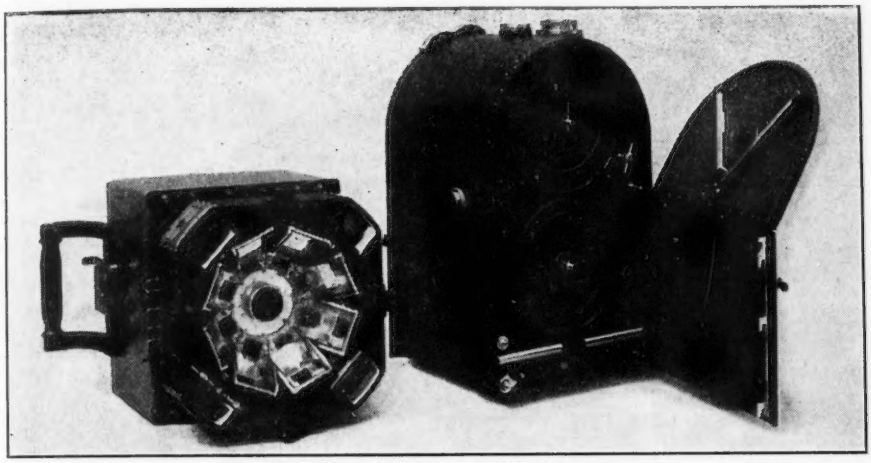

FIG. 18.—NINE-LENS CAMERA AND MAGAZINE.

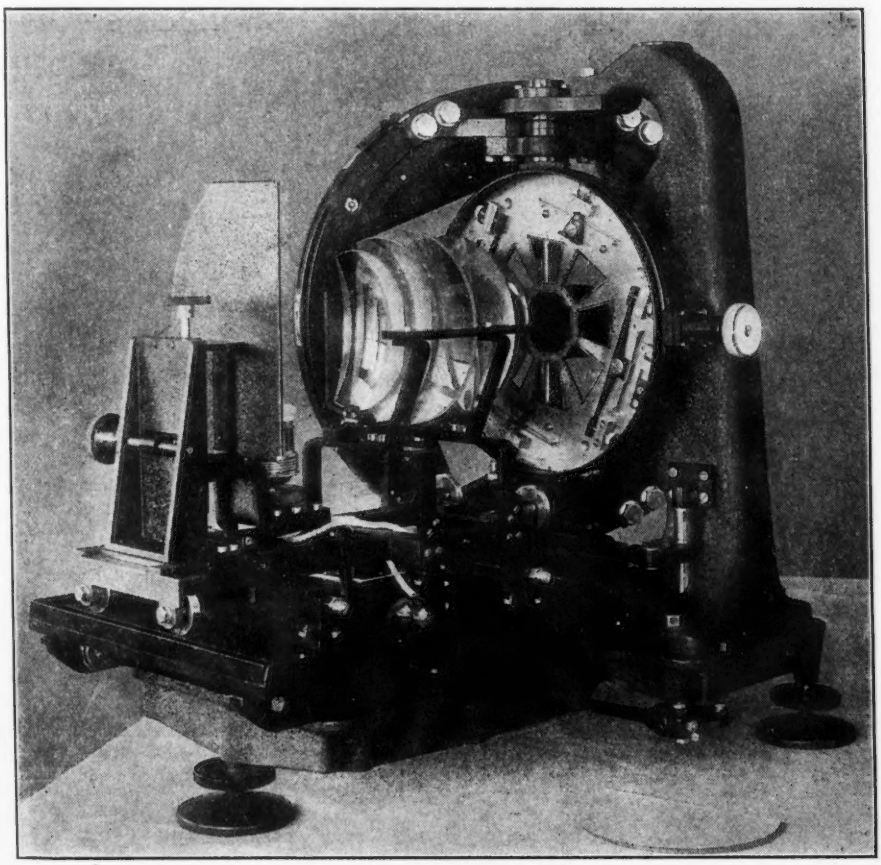

FIG. 19.—THE TRANSFORMER.

FIG. 20.—UNTRANSFORMED NINE-LENS PHOTOGRAPH, SCALE 1 : 60 000.

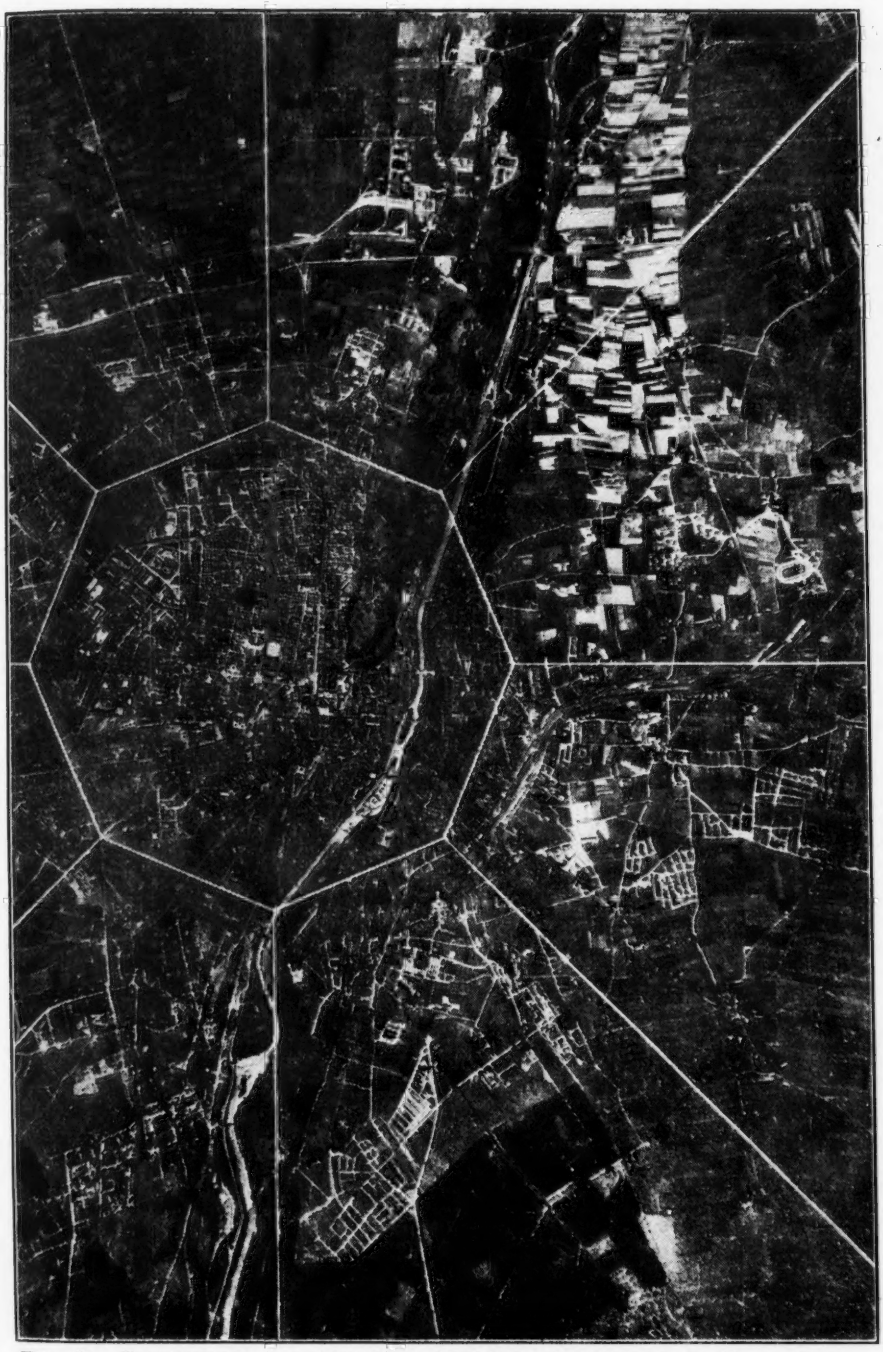

FIG. 21.—TRANSFORMED AND ASSEMBLED NINE-LENS PHOTOGRAPH OF SAME AREA AS SHOWN ON FIG. 20, SCALE 1 : 90 000.

that are observed through a stereoscope by means of the floating mark. First, the index mark of the left eye-piece of the stereoscope is brought to coincidence with the principal point in the left picture. Then the right picture is moved until the floating mark observed by both eyes touches the apparent ground. To mark this point on the right-hand picture the needle that coincides exactly with the index mark of the right eye-piece is moved down and the point on the ground below the image of the principal point is marked permanently on the plate. Similarly, any other point identified by a pair of images is marked permanently on both plates. This is a special advantage of the radial triangulator over other similar instruments, because even on very even ground the point looms up like a cairn. Micrometer screws are provided for different movements.

The next step is to determine the direction from the principal point of each plate to the images of the triangulation points shown on the plate. Each plate of an overlapping pair is adjusted in its plate-holder so that its center (principal point) coincides perfectly with the rotating center of the holder. This adjustment may be made by means of one of the eye-pieces of the double microscope or by means of a special microscope. The stereoscopic image of each triangulation point is then brought to coincidence with the floating mark by turning one plate-holder and manipulating one side-wheel at the same time. When this operation is complete, the angle to this image from the principal point of each plate is read to minutes. The same general process can be done graphically.

Stereoscopic fusion of these pictures reveals a surprisingly sharp relief due to the long base. With an overlap of 65%, the proportion of air-base line to height of camera is 5:3. The good results are due to the fact that both picture planes are parallel to each other and, at the same time, to the air-base line.

To check this method of image triangulation, Dr. Aschenbrenner made the following test: An area 30 km. long was selected, which had been covered by an accurate cadastral map on a scale of 1: 5 000, and in which there were a number of triangulation stations which could be accurately identified on the photographs. A strip of six pictures was taken and used as a traverse line, the centers of each plate being the traverse stations. A number of points identifiable on the cadastral map were intersected from adjacent traverse stations. In addition, a number of clearly visible points were chosen, located on either side of the traverse line. Intersection angles were measured from the traverse stations to these points. In this way overlapping trapezoids were formed over the entire traverse line. A method of computation was chosen such that each trapezoid was calculated by itself, assuming a length for the first side. Different methods may be used to refer the traverse line to the surface of the earth, the co-ordinates of two primary triangulation points at either end of the traverse line being known in the test. These two points were marked on the plates and were intersected from the traverse line, in order to enable the determination of its true position on the surface of the earth. The distance between the two points was then com-

puted from the photographic traverse on the assumed scale and on the earth, and, in this way, the scale of the traverse was obtained. By computing the azimuth of the connecting line between the two points, the orientation of the traverse line was obtained. In all, twelve control points were located on the cadastral map. A comparison between the computed co-ordinates of these control points and those taken from the map resulted in a mean error of 34 m. on the ground, which on a scale of 1:100 000 represents 0.034 mm.

In countries in which no triangulation network has been established, different methods may be applied. For example, on each end of the traverse line a base can be measured and its azimuth determined. A better method is to lay out two parallel traverse lines and connect them by photographic triangulation. To determine the bearing of such chains of triangles it would be sufficient to measure on the ground a base for each 100 km. In this way, with four base lines that could be checked or located by astronomical observations, a territory of about 10 000 sq. km. (3 600 sq. miles) could be surveyed. Dr. Aschenbrenner has reported[33] that a territory of 10 000 sq. km., in which only two known points were available, has already been surveyed by means of image triangulation, and good line maps prepared from the surveys.

Since 1930 Dr. Aschenbrenner has adapted his methods to the preparation of contour maps by stereoscopic plotting. The photographs are transformed to horizontal image planes by means of image triangulation. This may be done by calculation,[34] or by transforming the aerial photographs by means of the control points determined by image triangulation, at the same time reducing the aerial photographs to sizes that can be used for plotting in a space autograph. Thus, it is seen that stereoscopic pairs of transformed multiple-lens photographs may be plotted by either of the three space autographs mentioned at the beginning of this discussion. Even the stereometer or the stereo-goniometer, could be used.

At least six advantages may be stated for plotting small-scale contour maps from stereoscopic pairs of transformed panoramic photographs, in a space autograph: (1) Necessary adjustments can be made more quickly than by means of oblique pictures; (2) the optical relief of the stereoscopic model is accentuated and magnified as many times as the focal distance of the transformed reduction is contained in the focal distance of the space autograph; therefore, depth and height in the stereoscopic model are observed to an exaggerated scale, so to speak, so that contours can be drawn with greater accuracy even on flat terrain; (3) since the field of a wide-angled stereoscopic pair is many times greater than that produced by any other aerial camera, the plotting can be done much more quickly; (4) the area included in each exposure, is many times greater than that of an ordinary aerial camera; therefore, the time required to change the stereoscopic pairs is

[33] *Internationales Archiv für Photogrammetrie*, Vol. VII, II Band, 1931; a short informal report about his paper read at the Third International Congress of Photogrammetry, in Zurich, Switzerland, September 5–10, 1930.

[34] "Hohenkarten aus weitwinkligen Luftaufnahmen," von Dr. S. Finsterwalder, *Internationales Archiv für Photogrammetrie*, Vol. VII, 2d Pt. 1930–31.

greatly reduced; (5) for the foregoing reasons the number of control points is reduced considerably; and (6) the multiple-lens photographs are already generalized, while in the other processes the first map is the product of large-scale picture pairs and, therefore, must be redrawn to produce a generalized map on the scale of 1:15 000, or smaller, just as if plane-table sheets prepared on a scale of 1:15 000 were used to produce a final map on a scale of 1 in. equals 1 mile, or smaller. Every topographer knows what that means in terms of reproductions, redrawings, etc., because it takes time and personnel and reduces the exactness of the original map.

It is to be emphasized that by means of the panoramic camera and auxiliary equipment, areas as large as 400 to 500 sq. km. (144 to 180 sq. miles) can be plotted from one stereoscopic pair of photographs in comparison with 20 sq. km. that are obtained as a maximum by means of other aerial cameras.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# STRESSES IN REINFORCED CONCRETE DUE TO VOLUME CHANGES

## Discussion

BY MESSRS. WALTER DREYER, RALPH E. SPAULDING, A. H. BEYER AND A. G. SOLAKIAN, AND LESTER L. MEYER

WALTER DREYER,[13] M. AM. SOC. C. E. (by letter).[13a]—The writer is interested in Mr. Vetter's conclusion that from 0.60 to 0.75% of reinforcement is required in reinforced concrete in order to prevent unsightly cracks or water leaks. The Pacific Gas and Electric Company has constructed a number of reinforced concrete flumes and the prevention of unsightly cracks in these structures has always been one of the design problems. The earlier installations (1918 to 1920) followed the recommendation[14] of the Joint Committee on Concrete and Reinforced Concrete, which called for a transverse reinforcement of 0.3 per cent. These structures developed cracks at intervals, more or less regular, of 20 to 25 ft. These cracks are sufficient in size to permit water to seep through with consequent frost or ice action tending to increase the size of the cracks.

About 1926 preliminary studies were made on a hydro-electric project to be constructed on the Mokelumne River. These studies soon indicated that about 20 miles of reinforced concrete flume with a capacity of 550 cu. ft. per sec., would be constructed. About this time it also became necessary to construct a section of flume of nearly the same size and 1 600 ft. in length on another of the Company canals. It was decided to install varying quantities of longitudinal reinforcement in this flume in order to ascertain experimentally the amount of steel required to give satisfactory results. The information obtained from the experiment was to be used as a basis for the design of the longer Mokelumne flume.

The flume is 10 ft. wide by 7 ft. deep, the side-walls and floor having an average thickness of 6 in. The 1 600-ft. length was divided into four equal sections, each with a different quantity of longitudinal steel. The first section

NOTE.—The paper by C. P. Vetter, Assoc. M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1932, by Messrs. Searcy B. Slack, Charles S. Whitney, and M. Hirschthal.

[13] Asst. Chf., Div. of Civ. Eng., Pacific Gas & Elec. Co., San Francisco, Calif.

[13a] Received by the Secretary April 26, 1932.

[14] *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1140.

has 0.27%; the second, 0.31%; and third, 0.47%; and the fourth, 0.65%; no expansion joints were provided. Before water was turned in, the shrinkage cracks were marked and stationed. During the first part of September, 1928, the section was inspected while carrying 500 sec-ft. of water and the comparative size of the cracks and the amount of leakage were noted.

At that time, the section having 0.25% longitudinal wall reinforcement showed cracks which developed leaks sufficient in size to cause water to drip from a shoulder left by the outside forms at the bottom of the wall. These main cracks were spaced in general from 15 to 25 ft. apart. In the section having 0.31% reinforcement, the main cracks were spaced in general from 15 to 35 ft. apart, the leaks were smaller, and, in many cases, the wall was only damp to the bottom. In the section having 0.47% wall reinforcement, the main cracks averaged about 25 ft. apart, and the wall in the majority of cases was only wet or damp. In the section having 0.65% wall reinforcement, the cracks were very small and were spaced 75 to 100 ft. apart.

Since the quantity of water diminished to a small flow before the advent of cold weather, no observation was made under winter conditions. On May 22, 1929, an observation was made with more than 500 sec-ft. in the canal, the water temperature being 41° Fahr. It was observed that in the section having 0.27% longitudinal reinforcement, the leaks in general had increased and one in particular (which appeared to be at the end of a day's pour) had reached the proportions of approximately 0.5 gal. per min. Several leaks showed efflorescence and, in one case, disintegration, which was probably due to ice and frost during the winter. An increase in leaks and damp spots was also observed in the section having 0.31% reinforcement, although to a lesser degree. In both the aforementioned sections the amount of leakage was considered undesirable for such structures as reinforced concrete flumes when severe winters prevail. The leakage in the first section could endanger certain types of soil foundations, where bench-flumes are concerned.

The last two sections, having 0.47% and 0.65% reinforcement, showed satisfactory conditions, outside of an increase in number of sweat spots. The last section has few surface cracks for this type of construction.

As a result of this test it was concluded that at least 0.65% of longitudinal steel was required for satisfactory construction. This conclusion, arrived at entirely by experiment, is almost identical with that reached by Mr. Vetter as the result of a theoretical study.

The Mokelumne flume, as a result of the foregoing information, was designed and constructed with contraction joints at intervals of 20 ft. This construction, using about 0.2% longitudinal steel between the joints, which were made with a copper water-stop, proved to be less expensive than providing the quantity of longitudinal steel required to prevent cracking.

RALPH E. SPAULDING,[15] M. AM. SOC. C. E. (by letter).[15a]—Volume changes in reinforced concrete, particularly with reference to building structures, have interested the writer for more than twenty years. The only volume

[15] Cons. Engr., Jacksonville, Fla.

[15a] Received by the Secretary May 2, 1932.

change in reinforced concrete building structures that is worthy of attention as disturbing the respective computed stresses in the concrete and steel is the change of shrinkage due to drying. The amount of such shrinkage is somewhat variable, depending upon many factors, but its average is about 0.00035. In many cases it is considerably greater. The shrinkage of the structure between expansion joints (or over the structure as a whole if there are no expansion joints) if there is no steel reinforcement would be the amount of the shrinkage factor (0.00035) times the distance between expansion joints.

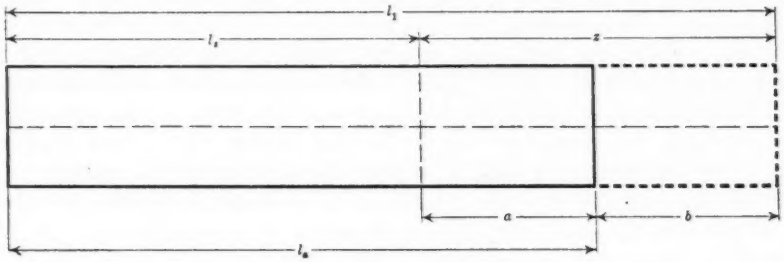


FIG. 7.

When steel reinforcement is introduced the shortening becomes a factor of the percentage of steel, the elasticity of the concrete, and the elasticity of the steel.

This is best illustrated, perhaps, by reference to Fig. 7. In addition to Mr. Vetter's notation, let

$l_1$ = original unit of length as cast = 1.0;
$l_s$ = length the unit would assume if there were no steel to resist the shrinkage = 1 — z = 0.99965;
$l_a$ = length actually assumed by the reinforced unit when shrinkage is resisted by the steel;
$a$ = induced stretching in the concrete;
$b$ = induced shortening in the steel;
$f_{cz}$ = unit stress in concrete due to the induced stretching; and,
$f_{sz}$ = unit stress in steel due to induced shortening.

The actual unit shortening of the reinforced concrete, $b$, may be computed by:

$$b = \frac{\frac{z}{np}}{1 + \frac{1}{np}} \qquad (39)$$

and the unit stretching of the concrete by,

$$a = z - b \qquad (40)$$

The comparative values of $f_{cz}$ and $f_{sz}$ are not a direct factor of $n$, as Mr. Vetter seems to indicate by his Equation (1), but varies with $p$ and the respective elasticities as well. For example, assume that $A_c = 200$, $A_s = 1$, and $n = 10$. From Equation (39), $b = 0.0003333$; and from Equation (40), $a = z - b = 0.00001667$. Then, $f_{cz} = a E_c = 50$ lb. per sq. in. tension, and $f_{sz} = b E_s = 10\,000$ lb. per sq. in. compression

To study its practical application in a structure, assume a structural column, 12 in. square, having eight $\frac{1}{2}$-in. square reinforcement bars. In that case, $A_c = 144$, $A_s = 2$, and $p = 0.01389$. Assume the breaking strengths of concrete = 2 500 lb. per sq. in., $E_c = 2\,500\,000$, $n = 12$, and $z = 0.00035$. By Equations (39) and (40), $b = 0.0003$ and $a = 0.00005$, respectively; $f_{cz} = a E_c = 125$ lb. per sq. in. tension, and $f_{sz} = b E_s = 9\,000$ lb. per sq. in. compression. Therefore, at no load the column steel is pre-stressed to 9 000 lb. per sq. in. and the concrete is negatively stressed to 125 lb. per sq. in. Beyond this point the changes of stress in the concrete and steel, due to loadings, are directly proportionate to $n$.

The load on the column necessary to bring the concrete stress down to zero would be $f_{cz}\ (A_c + n A_s) = 125\ (144 + 12 \times 2) = 21\,000$ lb. total load, and, at this point, the stress in the steel would be, $f_{sy} + 125\,n = 9\,000 + (125 \times 12) = 10\,500$ lb. per sq. in.

If the total load on the column (according to design) is 100 000 lb., the stress in the concrete would become 470 lb. per sq. in. and the total final stress in the steel, 16 143 lb. per sq. in., whereas as computed by ordinary methods, without regard to shrinkage, the computed unit stresses would have been 595 and 7 143, respectively.

As another example, consider a round column having eight $\frac{3}{4}$-in., round, vertical reinforcement bars, and in which $A_c$=113, $A_s$=3.536, and $p$=0.0313. Assume that $n = 10$, $z = 0.00035$, and the breaking strength of concrete is 3 000 lb. per sq. in. The column was originally designed for a total load of 178 000 lb., or with $f_c = 1\,200$ lb. per sq. in. and $f_s = 12\,000$ lb. per sq. in. (neglecting shrinkage in the concrete). The true stresses due to the inevitable shrinkage in the concrete are found by Equations (39) and (40) to be $b = 0.000266$ and $a = z - b = 0.000084$. As before, $f_{cz} = a E_c = 252$ lb. per sq. in. induced tension, and $f_{sz} = b E_s = 7\,980$ lb. per sq. in. induced compression.

Applying the computed load of 178 000 lb., this load would produce stresses (exclusive of the shrinkage stresses) of 1 200 lb. per sq. in. in the concrete and 12 000 lb. per sq. in. in the steel. Therefore, the true stresses which exist in the column as actually built are: $f_c = 1\,200 - f_{cz} = 1\,200 - 252 = 948$ lb. per sq. in., and $f_s = 12\,000 + f_{sz} = 12\,000 + 7\,980 = 19\,980$ lb., per sq. in., the actual stress in the concrete being only about 80% of the computed stress by ordinary formulas, and the actual stress in the vertical steel being about 165% of the computed stress.

It is evident then that the shrinkage in the concrete produces great changes in the computed stresses in both the concrete and the steel, reducing the concrete stresses and increasing the steel stresses. Furthermore, the shrinkage value of 0.00035 is by no means unusual, often it is much greater; certain it is that there is always some shrinkage of the concrete in a building structure, which produces approximately the foregoing results. Many designers are anxious to pre-stress the vertical steel in reinforced columns, when as a matter of fact the vertical steel is always pre-stressed, this being taken care of by the nature of the concrete itself.

This same shrinkage of concrete takes place, of course, in the beams and slabs of the structure, as well as in the columns, but its results are more complicated. Consider a typical floor-slab, as shown in Fig. 8, in connection with the following assumptions: 2 000-lb. concrete; $E_c = 2\,000\,000$; $n = 15$;

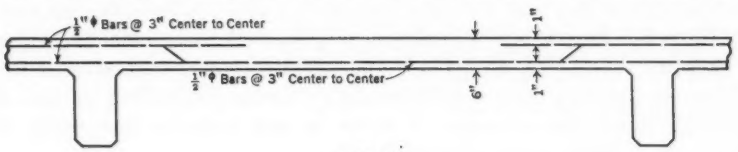


FIG. 8.

$w$ = load = 325 lb. per sq. ft.; $l$ = span = 12 ft.; and $W$ = total load per foot of width = 3 900 lb. The slab is continuous over many spans.

With such a floor-slab (neglecting shrinkage of concrete), considering the reinforcement as shown, the true bending moments are somewhat less than $\frac{1}{24}\ Wl$ between supports and somewhat more than $-\ \frac{1}{12}\ Wl$ over the supports, being about $0.0365Wl$ and $-0.0895Wl$, respectively.[16] Still assuming no shrinkage in the concrete the computed maximum stresses in the concrete and steel at the total load of 325 lb. per sq. ft. would be about as follows: Between supports, $f_c = 350$, and $f_s = 6\,250$; over supports, $f_c = 675$, $f_s = 16\,000$, and $f'_s = 5\,300$.

Neglecting all loading for the time being, and computing the stresses due to shrinkage, taking the shrinkage factor as 0.00035, and using Equations (39) and (40), it is found that over the supports the concrete is stressed to 172 lb. per sq. in. in tension and the steel (both upper and lower layers) is stressed to 7 900 lb. per sq. in. in compression. Between the supports—since the steel is all in one face, and, therefore, eccentric with the center line of tension of the concrete as it shrinks—the concrete usually ruptures on the upper face, either in minute "hair"-cracks, or, if there has been a construction joint in the center of the span, by a definite single crack easily observed. If no rupture takes place the stresses in the concrete and steel, by Equations (39) and (40), will be approximately, for concrete, 98 lb. per sq. in. tension, and for steel, 9 000 lb. per sq. in. compression, which is the reverse of the condition for which it was designed. In either event the slab between supports is unable to resist any positive moment until it has deflected sufficiently to overcome the shrinkage in the upper face.

As load is applied to the slab it is resisted at first only by the cantilever action over the supports. At these points the moment couple of the steel in the upper part, and both the steel and the concrete in the lower part, carry the entire load until the slab deflects sufficiently to overcome the shrinkage in the compressive concrete, and until that takes place the bending moments are: Over the supports, $M = -\frac{1}{8}\ Wl$; and between the supports, $M = 0$. The moments continue, as stated, until the concrete computed for compression is brought into bearing. For all practical purposes this is accomplished when

[16] See curves for bending moments in concrete beams with variable moments of inertia, Concrete Engineers' Handbook, Hool and Johnson, N. Y., 1918, Article 54, pp. 339–341.

the induced tensile stress in the compression concrete over the supports ($f_{cz}$) is reduced to 0. Since this concrete (normally in compression) now has an induced tension of 172 lb. per sq. in., the load necessary to overcome that would be the amount of loading which would normally (not considering shrinkage) stress the concrete over the supports to 172 lb. per sq. in. at a bending moment of $-\frac{1}{8}\ Wl$, or computing by regular formulas, $W$ = about 680 lb. ($w$ = about 56 lb. per sq. ft.). At this time the added stress in the compressive steel over the supports is about 1 310 lb. per sq. in.

Beyond this point, and up to the total loading of $W = 325 \times 12 = 3\,900$ lb., or an added loading of $3\,900 - 680 = 3\,220$ lb., the additional stresses would be those computed by using the then true bending moments of $-0.0895\ Wl$ over supports and $0.0365\ Wl$ between supports. These additional unit stresses are approximately: Over the supports, $f_s = 13\,200$; $f_c = 580$; and $f'_s = 4\,370$; and between supports, $f_s = 5\,150$; and $f_c = 290$.

At full loading the stresses in the tensile steel both over and between the supports are sufficiently great so that the concrete on these faces can no longer carry tension, and under full load any tensile stresses in the concrete in these faces can be eliminated as is regularly the practice in such calculations. Combining the stresses obtained the results are as indicated in Table 2.

TABLE 2.—FULL LOAD STRESSES IN CONCRETE AND STEEL, IN POUNDS PER SQUARE INCH

| Stress symbol | Stress due to shrinkage of concrete | Stress due to enough load to bring concrete into compressive bearing | Stress due to remainder of load | Total final unit stress |
|---|---|---|---|---|
| Over supports: | | | | |
| $f_s$ | ..... | −3 900 | −13 200 | −17 100 |
| $f_c$ | −172 | +172 | +580 | +580 |
| $f'_c$ | +7 900 | +1 290 | +4 370 | +13 560 |
| Between supports: | | | | |
| $f_s$ | ..... | ..... | −5 150 | −5 150 |
| $f_c$ | ..... | ..... | +290 | +290 |

Table 3 is a summary of all the cases as given, all of which are typical for any building structure.

It must be remembered that shrinkage always takes place in a reinforced concrete building structure, and produces the following results:

(*a*) It reduces the stresses in the concrete, usually from 15% to 25% less than the concrete stresses as predetermined by ordinary methods.

(*b*) It always acts to pre-stress the compressive steel, the actual stress being usually more than twice the stress as ordinarily computed; it greatly increases the efficiency of compressive reinforcement.

(*c*) It increases the negative moments over supports, and, of course, decreases the positive moments, causing much more of the load to be carried by cantilever action.

(*d*) It increases, somewhat, the stresses in the tensile steel over supports.

Taken as a whole the changes in stress brought about by this shrinkage of the concrete are not dangerous; on the contrary, although often annoying, they are rather advantageous. This is particularly true in the case of columns. When the shrinkage action causes stresses in the cantilever steel

TABLE 3.—TYPICAL STRESSES IN CONCRETE AND STEEL FOR REINFORCED CONCRETE STRUCTURES

| Member | Stresses as computed, neglecting shrinkage of concrete | Approximate true stresses because of shrinkage of concrete | Approximate variation increase or decrease, percentage |
|---|---|---|---|
| Column No. 1: | | | |
| $f_s$ | +7 140 | +16 140 | +126 |
| $f_c$ | +595 | +470 | —21 |
| Column No. 2: | | | |
| $f_s$ | +12 000 | +19 980 | +66 |
| $f_c$ | +1 200 | +950 | —21 |
| Floor-Slab: | | | |
| Over supports: | | | |
| $f_s$ | —16 000 | —17 100 | +7 |
| $f_c$ | +675 | +580 | —14 |
| $f'_s$ | +5 300 | +13 560 | +156 |
| Between supports: | | | |
| $f_s$ | —6 250 | —5 150 | —17 |
| $f_c$ | +350 | +290 | —17 |

that are too high, those friends of ignorance—yield point and elastic recovery—step in and help to equalize the stresses. While this shrinkage is not a disadvantage in the ordinary or average case, nevertheless designing engineers should bear in mind that the shrinkage is inevitable, and there may be occasional cases in which to overlook its existence, would result in danger to the structure.

A. H. BEYER,[17] M. AM. SOC. C. E., and A. G. SOLAKIAN,[18] ASSOC. M. AM. SOC. C. E. (by letter).[18a]—The writers have made an attempt to check by photo-elastic methods[19] the theoretical analysis made by Mr. Vetter of the stresses in reinforced concrete due to volume changes. They have confined themselves to the simplest case, namely, the shrinkage stresses developed in a simple reinforced concrete member.

It was found that bakelite reinforced with aluminum rods has about the same relative physical properties as concrete reinforced with steel. The moduli of elasticity of aluminum and bakelite are, respectively, 675 000 lb. per sq. in., and 9 800 000 lb. per sq. in., giving a ratio of 14.5, compared with a ratio of 15 for concrete and steel. The coefficients of expansion at room temperature for bakelite and aluminum are about the same, namely, 0.000035 per 0° cent. Furthermore, the internal stresses developed in a specimen of

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[18] Lecturer, Mech. Eng., and Research Associate in Civ. Eng., Columbia Univ., New York, N. Y.

[18a] Received by the Secretary May 19, 1932.

[19] "A Treatise on Photo-Elasticity," by Dr. E. G. Coker and Dr. L. N. G. Filon, Cambridge Univ. Press, Lond., 1931.

bakelite reinforced with an aluminum rod during the processes of cooling and curing are qualitatively representative of the internal stresses developed in a reinforced concrete structure due to the shrinkage of the concrete.

The specimen analyzed consisted of a rectangular plate with a length three times its width and reinforced with an aluminum rod, 0.03 in. in diameter, all as shown in Fig. 9. It was made by pouring the molten

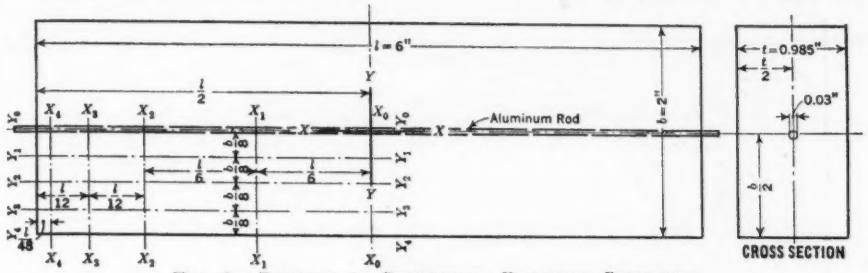


FIG. 9.—DETAILS OF REINFORCED BAKELITE SPECIMEN.

material into a metal form, $1\frac{1}{4}$ in. deep, $2\frac{1}{4}$ in. wide, and 6 in. long, which contained at its geometrical axis an aluminum rod. The specimen was cooled in about two days in a special curing oven to room temperature. The two flat surfaces were then machined and polished so as to secure a specimen of a uniform thickness of $1\frac{1}{8}$ in., with perfectly parallel faces.

An internal stress distribution in this specimen is believed to be similar to the internal stress distribution developed by shrinkage in a reinforced concrete member unrestrained at its ends and reinforced at its center by one longitudinal rod.

To determine the actual stress distribution, the specimen was placed in the path of circular polarized white light under which the color fringes were revealed. Fig. 10 shows the boundaries of the color fringes on one side of

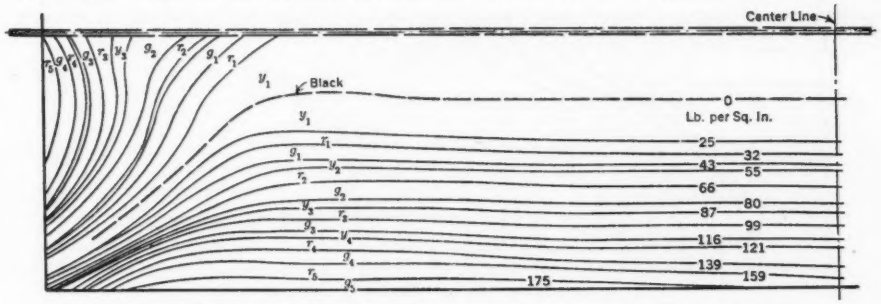


FIG. 10.—ISOCHROMATICS. LINES OF CONSTANT MAXIMUM SHEAR STRESS.

the middle transverse axis of the plate and for a thickness of specimen of 0.985 in. The colors—yellow, green, and red—are denoted by $y$, $g$, and $r$, respectively, and the subscript in each case denotes the order; thus, $r_5$ denotes fifth order red. In Fig. 10 each fringe line represents a contour along which the maximum shear intensity is constant. The evaluation of these stresses for each fringe line, technically known as isochromatic bands, was made by means of a test beam subjected to bending moments of known intensity.

The directions of the principal stresses were obtained from the isoclinic curves shown in Fig. 11. These curves were obtained by passing plane polarized white light through the specimen, using angular settings at intervals of 10° for the crossed Nicols,[20] always keeping the longitudinal axis of the specimen in a fixed horizontal position. For any given position of the Nicols, narrow black bands appeared, the so-called isoclinic bands, within which

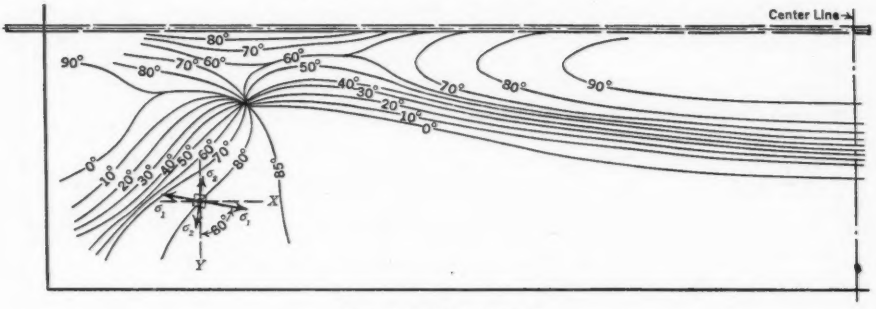


FIG. 11.—ISOCLINICS.

the direction of the principal stresses coincide with the axis of inclination of the Nicols.

The principal stress trajectories thus obtained from these isoclinics by graphical methods are noted in Fig. 12 which shows at each point the direction of the two principal stresses, $\sigma_1$ and $\sigma_2$, but not their intensities. In order to obtain sharper and better defined isoclinic lines the thickness of the original specimen was reduced by machining to 0.328 in.

Fig. 13 shows the isoclinic lines for a 20° inclination of the Nicols. Only the lines for the lower left quadrant are shown. The isoclinics for the upper

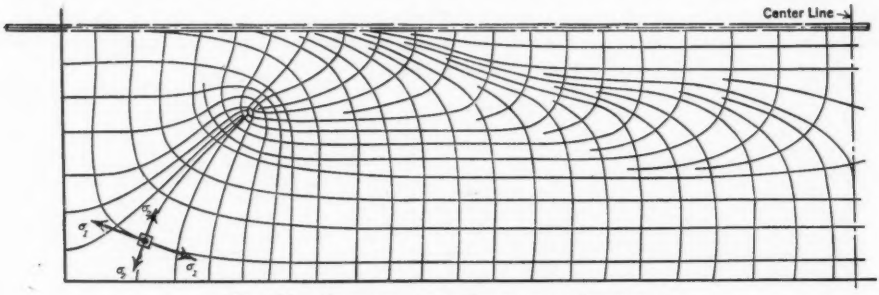


FIG. 12.—DIRECTIONS OF PRINCIPAL STRESSES.

left quadrant are the complementary values, that is, the same as for 70° isoclinics in the lower left-hand quadrant. This indicates that the stress trajectories are symmetrical about a centrally located rod. The "color fringes" mentioned in connection with Fig. 10 are reproduced in Fig. 14.

The intensities of the maximum shear stresses, that is, one-half the algebraic principal stress difference at the several transverse cross-sections of

[20] For a definition, see "A Treatise on Photo-Elasticity," p. 58.

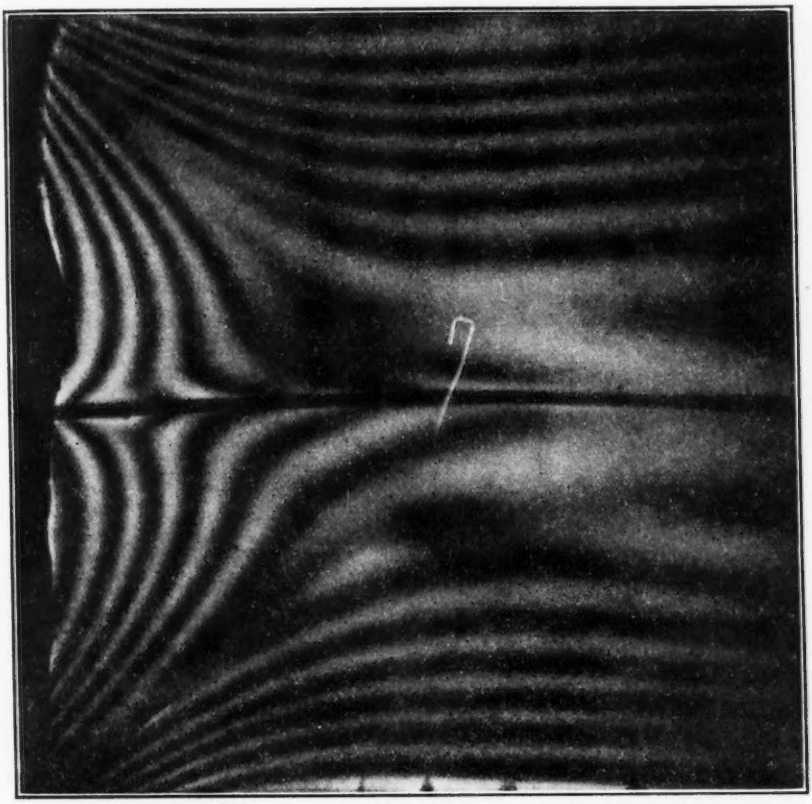

FIG. 13.—STRESS FRINGES IN A TEST SPECIMEN (TAKEN WITH CIRCULAR POLARIZED WHITE LIGHT).

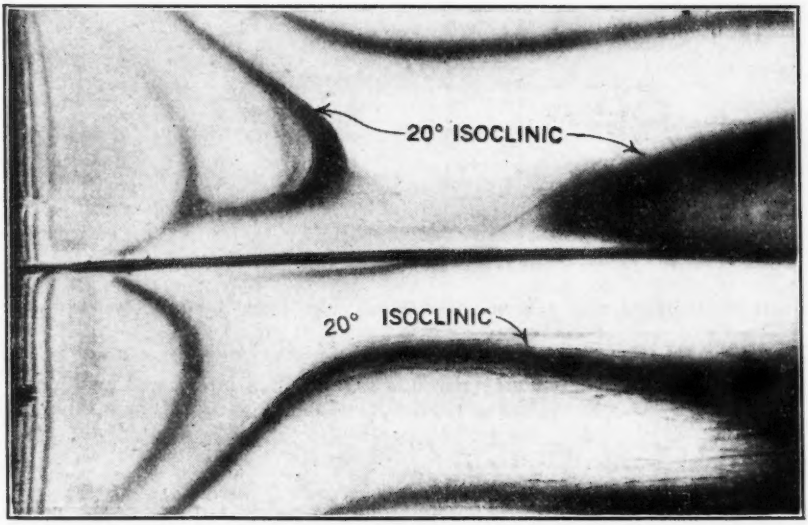


FIG. 14.—TWENTY-DEGREE ISOCLINICS (TAKEN WITH PLANE POLARIZED WHITE LIGHT).

the specimen, are shown graphically in Fig. 15. These values were obtained directly from the isochromatic lines of Fig. 10. The directions of these maximum shear stresses in Fig. 15 are at every point inclined at an angle of 45° to the principal stress trajectories shown in Fig. 12. The interesting fact to be observed is that the shear stress intensities reach their maximum values

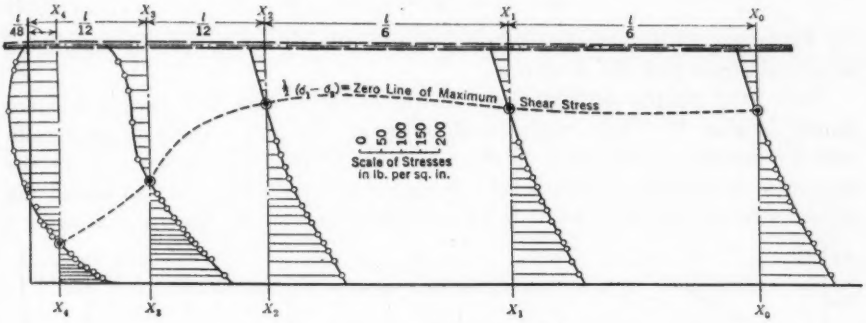


FIG. 15.—INTENSITIES OF MAXIMUM $\frac{P - p'}{2}$ STRESSES.

where the reinforcement enters the specimen and, also, at the outer longitudinal edges of the specimen.

The components of the principal stresses parallel and perpendicular to the axis of the reinforcing rod are computed from formulas based upon the assumption of a co-planor stress analysis. Although the actual stress distribution resulting from shrinkage is, strictly speaking, a three-dimensional one, it is believed that no large errors are introduced when the thickness of the specimen is relatively small compared to the width.

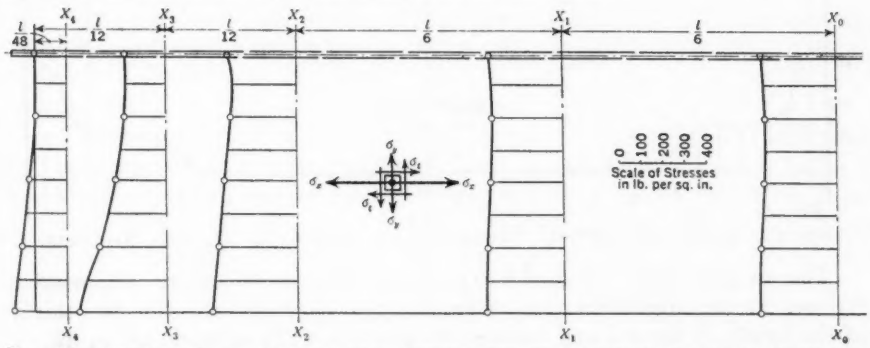


FIG. 16.—INTENSITIES OF LONGITUDINAL COMPONENTS OF MAXIMUM PRINCIPAL STRESSES.

Based upon a two-dimensional stress analysis, the Maxwell-Filon formula has been used to compute the stress intensity in the test specimen parallel to the axis of the rod, as follows:

$$\sigma_x = \sigma_{xo} - \int_{xo}^{x} \frac{\partial \tau}{\partial y}\, dx \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

in which, at any section, $\sigma_{xo}$, is a boundary stress parallel to the axis of the specimen. At this point, $\sigma_{yo}$ is equal to 0, and $\partial\tau$ is the rate of change in

the shear stress with respect to the $y$-axis. The shear stress, $\tau$, is computed from the formula, $\tau = (\sigma_1 - \sigma_2) \sin \phi \cos \phi$. The values of $\sigma_x$ have been evaluated at the various sections by graphical integration and its numerical values are shown in Fig. 16. The corresponding value for $\sigma_y$ may be computed from the following formula:

$$\sigma_y = \sigma_x - (\sigma_1 - \sigma_2) \cos 2\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

In Equation (42) $\phi$ is the angle between the direction of the maximum principal stress and the $X$-axis.

The final results of the photo-elastic analysis that has been made are shown on Fig. 17. The upper curve (Fig. 17($a$)), shows the average tensile stresses parallel to the axis of the specimen at the various sections in the specimen as a result of shrinkage. Note the short distance within which the tensile stresses, which amount to about 350 lb. per sq. in., become constant.

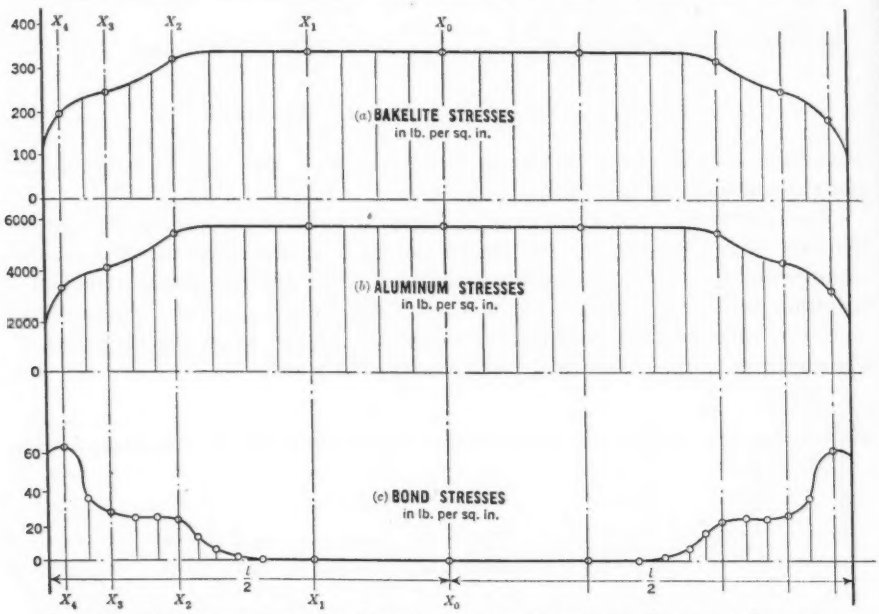


FIG. 17.—SHRINKAGE STRESSES DEVELOPED IN A REINFORCED BAKELITE SPECIMEN.

The middle curve (Fig. 17($b$)), shows the corresponding compressive stresses in the aluminum rod. This is practically constant for two-thirds of the length of the rod and amounts to approximately 6 000 lb. per sq. in.

The bond stresses shown in the lower curve (Fig. 17($c$)), reached the highest values at the ends, namely, about 65 lb. per sq. in., which gradually reduced to zero at about the quarter-points.

From the foregoing comments it is seen that the photo-elastic investigation made by the writers corroborated qualitatively the assumptions made by the author in computing by theoretical methods the maximum tensile stresses set up in the concrete, the compressive stresses in the steel, and the bond stresses between the steel and the concrete.

LESTER L. MEYER,[21] ASSOC. M. AM. SOC. C. E. (by letter).[21a]—The author's detailed analysis of this important subject is fundamental and complete. It is a subject that has been little discussed in this fashion in current engineering literature, although the method of design has been used by the Los Angeles Water Department for reinforcing concrete slabs on the face of earth dams. The design formulas used were those derived by Julian Hinds, M. Am. Soc. C. E., which are essentially the same as those given by the author, with the exception of the bond assumption which Mr. Hinds assumes to vary uniformly from a maximum at the crack to zero at the center of the block. This difference in the bond assumption, however, does not affect the only really useful formula for actual design, namely, Equation (12*a*), which prevents the metal from passing the yield point upon rupture of the concrete.

The Los Angeles Water Department made a series of tests to obtain the characteristics of slabs having different quantities of steel. Although the investigation is not complete, some interesting results have been obtained. These test slabs were 15 ft. in length and varied from 6 by 6 in. to 8 by 11 in. in cross-section. There was a continuous row of strain-gauge points on the top side, 10 in. apart. A loop of $\frac{3}{4}$-in. steel was cast in the ends of each specimen so that a tension load could be applied to stretch them. Each specimen was tested by stretching it with a hydraulic jack that had been calibrated on a Universal testing machine and was capable of reading the loads to the nearest 200 lb. The strain was measured with a Whittemore strain-gauge reading to 0.0001 in. The concrete used in the test had an average compressive strength of 2 800 lb. per sq. in. at 28 days, and a tensile strength that varied from 205 to 255 lb. per sq. in. at 28 days. The steel had a yield point varying from 45 000 to 58 000 lb. per sq. in. A brief summary of results is shown in Table 4.

This method of pulling the test specimen duplicated closely the conditions of a slab subject to temperature drop only, with the exception of a small amount of shrinkage that took place during the period before the specimen was tested.

As the slab was subjected to more and more tension, it expanded until at a unit expansion of from 0.00008 to 0.00011 the first crack occurred (this usually corresponded to a stress of about 200 lb. per sq. in. in the concrete). Application of additional load after the first crack had occurred, would never give a unit strain in an uncracked segment greater than this first strain just prior to the first crack. Additional load simply produced additional cracking and, hence, additional over-all expansion of the total length of the slab.

Those familiar with concrete are aware that its tensile strength is much more erratic than its compressive strength and that carefully made concrete may vary widely in tensile strength in a single batch. For this reason the first crack usually occurs at the place of the weakest concrete in tension, and as the load increases cracks occur successively at places of increasing ultimate tensile strength. The uncracked segments remaining are a case of the survival

[21] Asst. Civ. Engr., Bureau of Water-Works and Supply, Los Angeles, Calif.

[21a] Received by the Secretary May 23, 1932.

of the strongest concrete. Table 4 shows that specimens having the greatest steel ratios developed the greatest ultimate tensile strength of the concrete. This variation in tensile strength makes it impossible to compare results of tests with the author's theories, except in a very general way.

TABLE 4.—SUMMARY OF RESULTS OF TESTS ON CONCRETE TEST SLABS

| Specimen No. | Steel ratio | Number of bars | Size of bars, in inches | Value of $q$ | Age tested, in days | Maximum tensile strength developed in test, in kips | Maximum unit elongation of slab before reaching yield point |
|---|---|---|---|---|---|---|---|
| 7*b* | 0.25 | 2 | ¼ | 16.0 | 38 | 102 | 0.00010 |
| 6*b* | 0.35 | 2 | ¼ | 16.0 | 39 | 205 | 0.00027 |
| 1*b* | 0.50 | 1 | ½ | 8.0 | 40 | 240 | 0.00065 |
| 2*b* | 0.75 | 2 | ½ | 8.0 | 41 | 290 | 0.00086 |
| 3*b* | 1.00 | 1 | ⅞ | 6.4 | 42 | 260 | 0.00105 |
| 4*b* | 1.25 | 2 | ⅞ | 6.4 | 46 | 360 | 0.00108 |
| 5*b* | 1.50 | 2 | ⅞ | 6.4 | 45 | 394 | 0.00114 |
| 1*a* | 0.50 | 1 | ¾ | 5.3 | 53 | 202 | 0.00129 |
| 3*a* | 0.49 | 2 | ¾ | 8.0 | 53 | 223 | 0.00102 |
| 5*a* | 0.50 | 4 | ¾ | 10.7 | 53 | 227 | 0.00099 |
| 6*a* | 0.50 | 9* | ¼ | 16.0 | 53 | 241 | 0.00103 |

| Specimen No. | Temperature drop corresponding to maximum elongation, in degrees | Temperature drop from Equation (27), in degrees | Average opening of cracks at maximum temperature drop, in inches | Average distance between cracks before yield point was reached, in inches | Average distance between cracks after yield point, in inches | L min. from Equation(12*f*) (Table 1), in inches† |
|---|---|---|---|---|---|---|
| 7*b* | 15 | 110 | 0.0062 | 120 | 120 | 50 |
| 6*b* | 40 | 160 | 0.0122 | 70 | 70 | 36 |
| 1*b* | 96 | 137 | 0.0111 | 20 | 16 | 50 |
| 2*b* | 129 | 118 | 0.0208 | 22 | 15 | 33 |
| 3*b* | 157 | 84 | 0.0161 | 16 | 8 | 31 |
| 4*b* | 161 | 98 | 0.0210 | 22 | 9 | 25 |
| 5*b* | 171 | 95 | 0.0260 | 22 | 10 | 21 |
| 1*a* | 193 | 115 | 0.0310 | 25 | 20 | 75 |
| 3*a* | 150 | 127 | 0.0305 | 35 | 12 | 51 |
| 5*a* | 148 | 144 | 0.0248 | 30 | 10 | 37 |
| 6*a* | 153 | 138 | 0.0201 | 16 | 15 | 25 |

* Plain bars, all others deformed.
† Assuming unit tensile and bond strength equal.

Another indeterminate factor is the bond stress. D. A. Abrams, M. Am. Soc. C. E., found[22] that as the bar slips, the unit bond resistance increases and that in the case of deformed bars the bond stress may reach three or four times the tensile strength after considerable slip has taken place. For plain bars the maximum bond stress, which occurs approximately after a movement of 0.01 in., is more nearly equal to the tensile strength. These ideas, as well as the author's Equation (12*f*) (Table 1), are verified somewhat by comparing the results of the test with Equation (12*f*), as shown in Table 4, assuming the bond stress equal to the tensile strength. It is noticed that the plain bars used in the specimen, 6*a*, show the closest agreement with Equation (12*f*). The other specimens, in which deformed bars were used, indicate unit bond stresses from two to four times the tensile strength.

*Opening of Cracks.*—At rupture, the cracks opened from 0.004 to 0.02 in., the average being 0.01 in. On removing the load the crack would close from

[22] *Bulletin 71,* Univ. of Illinois Eng. Experiment Station.

50 to 70% of the amount it had opened; but in no case would it close entirely regardless of the amount of steel. In two instances invisible cracks occurred before the load was applied. At point of failure of the next crack, these cracks had opened only a small amount compared with those rupturing during stress. Probably a considerable part of the opening of the cracks is due to bond slip caused by impact stress at the time of rupture.

The writer believes that the amount the cracks open, is an important consideration. Obviously, it was neglected by the author because of the impossibility of treating it by pure mathematics. However, further experimental work might bring out the factors governing the width of cracks. By observing the summary of the results of the test, it is seen that although some specimens could resist a temperature drop of 150° Fahr., or more, the cracks produced would have widths of 0.03 in., or more. In hydraulic structures this would permit considerable water leakage; it would be unsightly, and, in some cases, dangerous. The maximum width of cracks that should be tolerated in a structure depends, of course, on its use. For hydraulic structures, a crack of more than 0.005 in. in width probably will not seal itself by autogenous healing and swelling as is commonly the case for fine cracks in new concrete hydraulic structures. If cracks could be forced to occur at more frequent intervals by moulding indentations in the slab at predetermined intervals, the cracks might be made smaller. The effect of such indentation would be virtually to increase the steel ratio, which (according to the author's formula) should produce cracks at more frequent intervals.

The writer believes that it is apparent from the results of the tests that an equation for maximum temperature drop should be based on a limiting stress in the steel at the crack, and not on the critical penetration of bond stresses into the uncracked segment; however, the tests also show that when Equation (12*a*) (Table 1), is satisfied, there is more than ample provision for temperature requirements.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# SUSPENSION BRIDGES UNDER THE ACTION OF LATERAL FORCES

## Discussion

BY MESSRS. E. L. PAVLO, RALPH A. TUDOR, F. H. CONSTANT, CHARLES DERLETH, JR., A. A. EREMIN, ELMER C. OSGOOD, AND CHARLES M. SPOFFORD AND JOHN B. WILBUR

E. L. PAVLO,[5] ESQ. (by letter).[5a]—Owing to the great length of spans in suspension bridges, the elastic behavior and the induced stresses in this type of structure under the action of lateral forces are of considerable importance. As shown by the examples in the paper, the wind pressure on such spans amounts to more than 1 000 lb. per lin. ft. of bridge and the resulting stresses are of a magnitude sufficient to affect the determination of the width of structure. In addition, as a considerable part of the applied pressure is transferred from the trusses to the cables and thence to the saddles, the design of the towers is also affected. It may be stated also that the sag ratio, and therefore the general dimensions, of the structure are dependent upon the wind stresses in as much as the lateral deflections are proportional to the sag ratio for a given length of span. It will thus be seen that the wind stresses play an important part in the design of suspension bridges and their rigorous analysis is fully justified.

In this respect, in connection with the present tendency toward increased spans, this paper is of timely importance. The authors are to be commended for the clear and concise presentation of the problem of wind distribution and for the practical methods developed for its solution. The paper also contains much valuable information concerning several existing bridges. The rather condensed nature of the paper affords the writer an opportunity to elucidate some of the remaining aspects of the problem, as developed by him,[5b] and to suggest three possible means of avoiding the procedure of suc-

NOTE.—The paper by Leon S. Moisseiff, M. Am. Soc. C. E., and Frederick Lienhard, Esq., was published in March, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings* as follows: May, 1932, by Messrs. P. L. Pratley, and Charles A. Ellis.

[5] Asst. Engr., Robinson & Steinman, New York, N. Y.

[5a] Received by the Secretary April 21, 1932.

[5b] See thesis submitted to Massachusetts Institute of Technology in partial fulfillment of the requirements for the degree of Doctor of Science.

cessive approximations in the elastic distribution method described by the authors:

(1) By deriving and solving the differential equation of the deflection curve of the stiffening trusses;
(2) By writing the deflection equations and solving them simultaneously; and,
(3) By first assuming that the pressure transferred from the trusses to the cables varies as a parabola.

In addition to the notation introduced by the authors, the following will be used:

Let, $H_a$ = horizonal component of cable stress before the advent of wind pressure, total for all cables.
$H_w$ = additional horizontal component of cable stress due to wind pressure.
$H = H_a + H_w$.
$w = w_t + w_c$.
$X$ = actual intensity of wind pressure sustained by trusses at any section.
$r$ = intensity of wind pressure transferred from trusses to hangers at any section, called restitution.
$Q$ = actual shear at any section of the truss.
$M$ = actual moment at any section of the truss.
$M_0$ = moment due $w_c$.
$M_w$ = moment due to $w$.
$M_c$ = moment due to $r + w_c$.
$u$ = transverse tower deflection.

(*1*).—*The Elastic Curve Method.*—Referring to Fig. 11(*a*), the moment at any section, *A-A*, is,

$$M = M_w - H\,\delta_{cx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

From Fig. 11(*c*), by taking moments around the cable, and noting that $\Delta_\delta = \delta_{tx} - \delta_{cx}$,

$$r\,(h_t - y)\,\cos\theta = p_{sx}\,(\delta_{tx} - \delta_{cx})$$

Substituting unity for cos $\theta$ (that is, by taking the length of hanger equal to its vertical projection, since the difference is negligible), solving for $\delta_{cx}$ and substituting $\delta_{cx}$ in Equation (15);

$$M = M_w - H\left[\delta_{tx} - r\,\frac{(h_t - y)}{p_{sx}}\right] \quad \ldots\ldots\ldots\ldots (16)$$

In Equation (16), in case of full loading,

$$M_w = w\,x\,\frac{l - x}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

By definition,

$$r = w_t - X \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The load, $X$, can be expressed in terms of $\delta_{tx}$ as its fourth derivative multiplied by $E\,I$; thus,

$$X = EI\,\frac{d^4\,\delta_{tx}}{dx^4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Substituting Equation (19) in Equation (18),

$$r = w_t - EI \frac{d^4 \delta_{tz}}{dx^4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

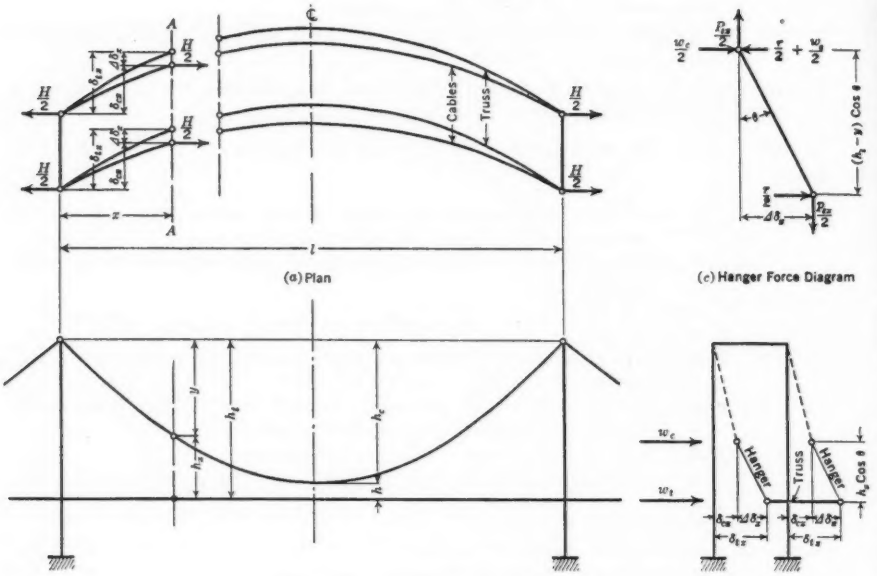


FIG. 11.—SUSPENSION BRIDGE UNDER THE ACTION OF LATERAL FORCES.

The equation of the cable curve referred to the origin at the top of the tower, is:

$$y = 4\, h_c\, x\, \frac{l - x}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Substituting $M_w$, $r$, and $y$ from Equations (17), (20), and (21) in Equation (16),

$$M = w\, x\, \frac{l - x}{2} - H \left\{ \delta_{tz} - \frac{1}{p_{sx}} \left( w_t - EI \frac{d^4 \delta_{tz}}{dx^4} \right) \left[ h_t - 4\, h_c\, x \frac{l - x}{2} \right] \right\} \quad \ldots (22)$$

In the differential notation, $M = -EI \frac{d^2 \delta_{tz}}{dx^2}$; hence Equation (22) becomes,

$$-EI \frac{d^2 \delta_{tz}}{dx^2} = w\, x\, \frac{l - x}{2}$$

$$- H \left\{ \delta_{tz} - \frac{1}{p_{sx}} \left( w_t - EI \frac{d^4 \delta_{tz}}{dx^4} \right) \left[ h_t - 4\, h_c\, x \frac{l - x}{2} \right] \right\} \quad \ldots\ldots (23)$$

which is the differential equation of the elastic curve of stiffening trusses. This equation is not an integrable type, but can be solved by the use of the series:

$$\delta_{tx} = A_0 + A_1 x + A_2 x^2 + \ldots + A_n x^n \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

$$\frac{d^2\,\delta_{tx}}{dx^2} = 2\,A_2 + 6\,A_3\,x + 12\,A_4\,x^2 + \ldots + n\,(n-1)\,A_n\,x^{n-2} \ldots\ldots (25)$$

and,

$$\frac{d^4\,\delta_{tx}}{dx^4} = 24\,A_4 + 120\,A_5\,x + 360\,A_6\,x^2 + \ldots$$
$$\ldots + n\,(n-1)\,(n-2)\,(n-3)\,A_n\,x^{n-3} \ldots\ldots\ldots\ldots (26)$$

To evaluate the coefficients of the series proceed as follows: Applying the boundary conditions, $\delta_{tx} = 0$, $\frac{d^2\,\delta_{tx}}{dx^2} = 0$, and $EI\,\frac{d^4\,\delta_{tx}}{dx^4}$, when $x = 0$, to Equations (24), (25), and (26), the initial coefficients, $A_0$, $A_2$, and $A_4$, are determined. The general method of finding the coefficients and the law of their formation is to substitute the series in Equation (23) and to equate the coefficients of the same powers of $x$ on both sides of the equation. A knowledge of the general coefficient formula and several of the initial coefficients is sufficient to determine any number of coefficients. With these known, the deflection, $\delta_{tx}$, can be determined with any desired precision by the use of a sufficient number of terms in Equation (24). The slope, moment, shear, and the load actually sustained by the trusses are found by taking the first, second, third, and fourth derivatives of $\delta_{tx}$, respectively, and multiplying them by *EI*.

Equation (23) was derived for the center span. It can be applied to side spans as well; the only change necessary is to replace $h_t$ by $x \tan \alpha$ in Equation (23), $\tan \alpha$ being the slope of the side cable chord.

From the theoretical viewpoint the method has the advantage that it is quite general and that all load functions can be expressed in terms of a single variable, $x$. In case of partial loading the application of the foregoing method becomes involved and in this case the following method will be found more advantageous to apply.

(*2*).—*Method of Deflections.*—Part of the wind pressure applied to the trusses is transferred in the form of unknown concentrated loads through the hangers to the cables. The structure is, therefore, indeterminate to the $n$th degree, $n$ being the number of hangers. To obtain an exact solution it is necessary, therefore, to use $n$ independent equations with $n$ unknowns. In case of a symmetrically loaded structure the number of unknowns and equations will be $\frac{n}{2}$, since the unknowns at the panels equi-distant from the center will be equal. For practical purposes, however, it is unnecessary to obtain the distribution of wind pressure at every panel point. Owing to the regularity of the distribution curve it suffices to determine its ordinates at less frequent intervals. For this reason the span may be divided into a smaller number of panels thereby greatly reducing the number of the required equations.

In the present analysis the span is divided into ten equal panels and the applied pressure extends over the entire span. The indeterminacy of the structure is thus reduced to the fifth degree, necessitating the use of five

equations. These equations will be written in the form, $\delta_{tx} = \delta_{cx} + \Delta\,\delta_x$ (see Fig. 11(*d*)).

Let $X_1$, $X_2$, $X_3$, $X_4$, and $X_5$, be the actual intensity of wind pressure sustained by the lateral system at Panel Points 1, 2, 3, 4, and 5, respectively. The truss and cable loading diagrams will then be as shown in Figs. 12(*a*)

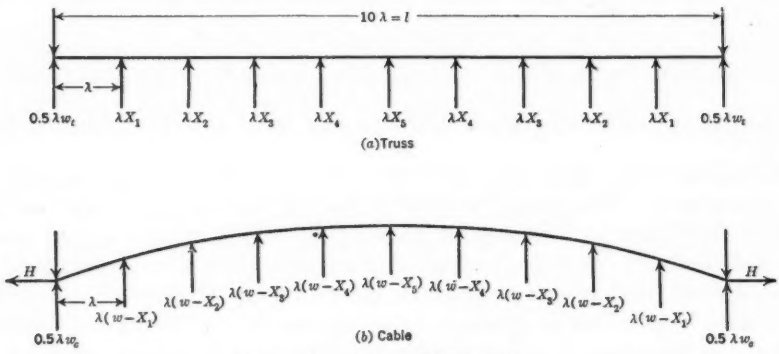


FIG. 12.—LOADING DIAGRAMS.

and 12(*b*). The loading diagram for the cable is shown for all cables combined.

Deflection $\delta_{tx}$ will be found by means of the moment-area principle as demonstrated in Tables 12 and 13.

TABLE 12.—WIND LOADS, SHEARS, AND MOMENTS

| Section | Load | Shear, $Q$ | Moment, $M$ |
|---|---|---|---|
| 0 | | | 0 |
| 0.1*l* | $\lambda X_1$ | $\lambda(X_1 + X_2 + X_3 + X_4 + 0.5X_5)$ | $\lambda^2(X_1 + X_2 + X_3 + X_4 + 0.5X_5)$ |
| 0.2*l* | $\lambda X_2$ | $\lambda(X_2 + X_3 + X_4 + 0.5X_5)$ | $\lambda^2(X_1 + 2X_2 + 2X_3 + 2X_4 + X_5)$ |
| 0.3*l* | $\lambda X_3$ | $\lambda(X_3 + X_4 + 0.X5_5)$ | $\lambda^2(X_1 + 2X_2 + 3X_3 + 3X_4 + 1.5X_5)$ |
| 0.4*l* | $\lambda X_4$ | $\lambda(X_4 + 0.5X_5)$ | $\lambda^2(X_1 + 2X_2 + 3X_3 + 4X_4 + 2X_5)$ |
| 0.5*l* | $\lambda X_5$ | $\lambda(0.5X_5)$ | $\lambda^2(X_1 + 2X_2 + 3X_3 + 4X_4 + 2.5X_5)$ |

TABLE 13.—MOMENT AREAS AND TRUSS DEFLECTIONS

| Section | Moment area | Deflection, $\delta_{tx}$ | Equation No. |
|---|---|---|---|
| 0 | | 0 | |
| 0.1*l* | $\lambda^3(4.5X_1 + 8X_2 + 10.5X_3 + 12X_4 + 6.25X_5)$ | $\frac{\lambda^4}{EI}(4.5X_1 + 8X_2 + 10.5X_3 + 12X_4 + 6.25X_5)$ | (27) |
| 0.2*l* | $\lambda^3(3.5X_1 + 7X_2 + 9.5X_3 + 11X_4 + 5.75X_5)$ | $\frac{\lambda^4}{EI}(8X_1 + 15X_2 + 20X_3 + 23X_4 + 12X_5)$ | (28) |
| 0.3*l* | $\lambda^3(2.5X_1 + 5X_2 + 7.5X_3 + 9X_4 + 4.75X_5)$ | $\frac{\lambda^4}{EI}(10.5X_1 + 20X_2 + 27.5X_3 + 32X_4 + 16.75X_5)$ | (29) |
| 0.4*l* | $\lambda^3(1.5X_1 + 3X_2 + 4.5X_3 + 6X_4 + 3.25X_5)$ | $\frac{\lambda^4}{EI}(12X_1 + 23X_2 + 32X_3 + 38X_4 + 20X_5)$ | (30) |
| 0.5*l* | $\lambda^3(0.5X_1 + X_2 + 1.5X_3 + 2X_4 + 1.25X_5)$ | $\frac{\lambda^4}{EI}(12.5X_1 + 24X_2 + 33.5X_3 + 40X_4 + 21.5X_5)$ | (31) |

Cable deflections at any point may be obtained by taking moments at the cable loads about the vertical axis passing through the point and dividing by $H$; thus:

$$\delta_{tc} = \frac{M_c}{H} = \frac{M_w - M}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Applying Equation (32) for the loading shown in Fig. 12($b$) to every tenth point of the span, the following deflection equations are obtained:

At Section 0.1$l$:

$$\delta_{cx} = \frac{\lambda^3}{H} (4.5w - X_1 - X_2 - X_3 - X_4 - 0.5X_5) \quad \ldots\ldots\ldots (33)$$

At Section 0.2$l$:

$$\delta_{cx} = \frac{\lambda^2}{H} (8w - X_1 - 2X_2 - 2X_3 - 2X_4 - X_5) \quad \ldots\ldots\ldots (34)$$

At Section 0.3$l$:

$$\delta_{cx} = \frac{\lambda^2}{H} (10.5w - X_1 - 2X_2 - 3X_3 - 3X_4 - 1.5X_5) \quad \ldots\ldots\ldots (35)$$

At Section 0.4$l$:

$$\delta_{cx} = \frac{\lambda^2}{H} (12w - X_1 - 2X_2 - 3X_3 - 4X_4 - 2X_5) \quad \ldots\ldots\ldots (36)$$

At Section 0.5$l$:

$$\delta_{cx} = \frac{\lambda^2}{H} (12.5w - X_1 - 2X_2 - 3X_3 - 4X_4 - 2.5X_5) \quad \ldots\ldots\ldots (37)$$

Hanger deflections can be found by taking moments of hanger loads about the cable. From Fig. 11($c$), and noting that $h_t - y = h_x$,

$$\Delta\delta_x = \frac{h_x}{p_{sx}} (w_t - X) \quad \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Applying the relation, $\delta_{tx} = \delta_{cx} + \Delta\delta_x$, to every tenth point as far as the center of the span, using the deflections as given by Equations (27) to (31), (33) to (37) and (38) for the five points of the half span, the following five equations with five linear unknowns are obtained:

$$\frac{\lambda^4}{EI} (4.5X_1 + 8X_2 + 10.5X_3 + 12X_4 + 6.25X_5)$$

$$= \frac{\lambda^2}{H} (4.5w - X_1 - 2X_2 - 2X_3 - 2X_4 - 0.5X_5) + \frac{h_1}{p_s} (w_t - X_1) \ldots (39)$$

$$\frac{\lambda^4}{EI} (8X_1 + 15X_2 + 20X_3 + 23X_4 + 12X_5)$$

$$= \frac{\lambda^2}{H} (8w - X_1 - 2X_2 - 2X_3 - 2X_4 - X_5) + \frac{h_2}{p_s} (w_t - X_2) \ldots\ldots (40)$$

$$\frac{\lambda^4}{EI} (10.5X_1 + 20X_2 + 27.5X_3 + 32X_4 + 16.75X_5)$$

$$= \frac{\lambda^2}{H} (10.5w - X_1 - 2X_2 - 3X_3 - 3X_4 - 1.5X_5) + \frac{h_3}{p_s} (w_t - X_3) \ldots (41)$$

$$\frac{\lambda^4}{EI}(12X_1 + 23X_2 + 32X_3 + 38X_4 + 20X_5)$$

$$= \frac{\lambda^2}{H}(12w - X_1 - 2X_2 - 3X_3 - 4X_4 - 2X_5) + \frac{h_4}{p_3}(w_t - X_4) \quad \ldots(42)$$

and,

$$\frac{\lambda^4}{EI}(12.5X_1 + 24X_2 + 33.5X_3 + 40X_4 + 21.5X_5)$$

$$= \frac{\lambda^2}{H}(12.5w - X_1 - 2X_2 - 3X_3 - 4X_4 - 2.5X_5) + \frac{h_5}{p_3}(w_t - X_5) \ldots(43)$$

The simultaneous solution of these equations furnishes the actual wind intensities sustained by the trusses at every tenth point of the span. With these known, shears, moments, and deflections can be readily evaluated. In the derivation of these equations, $I$ represents either a constant or equivalent moment of inertia.

In case it is desired to use a variable $I$, the only difference will be in finding truss deflections. Instead of finding the deflections by taking moments of moment areas and dividing them by $EI$, it will be necessary to use the moment of the $\frac{M}{I}$ area and divide it by $E$. The method of deflection is also applicable to partial wind loadings and the side spans. A procedure analogous

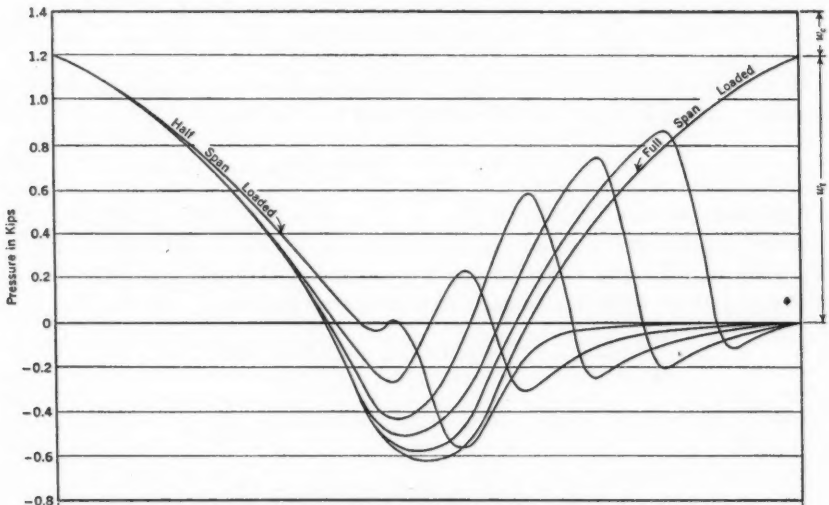


FIG. 13.—GEORGE WASHINGTON BRIDGE: DISTRIBUTION DIAGRAMS FOR FULL AND PARTIAL LOADINGS.

to the foregoing should be used in the derivation of equations. In both cases it is necessary to use a greater number of equations for the same accuracy on account of lack of symmetry in the loading and in the structure. Figs. 13, 14, and 15 show the results of applying the Method of Deflections to the

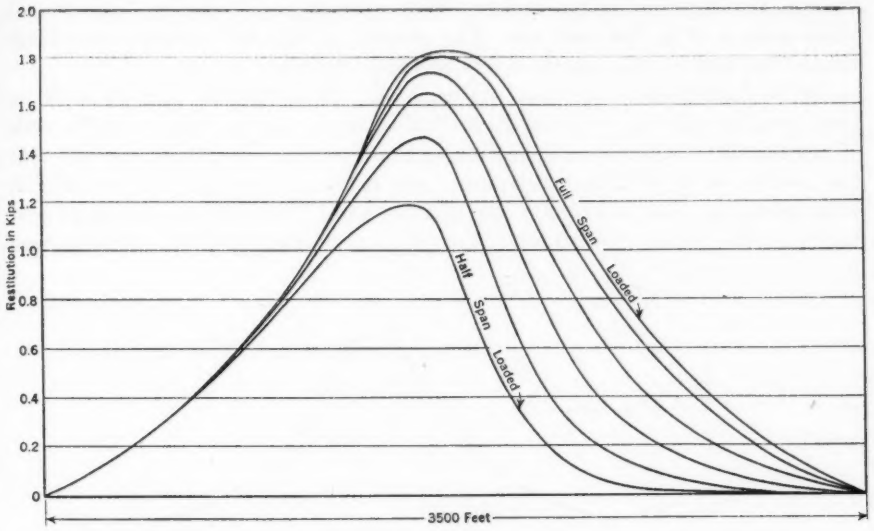


FIG. 14.—GEORGE WASHINGTON BRIDGE: RESTITUTION DIAGRAMS FOR PARTIAL WIND LOADINGS

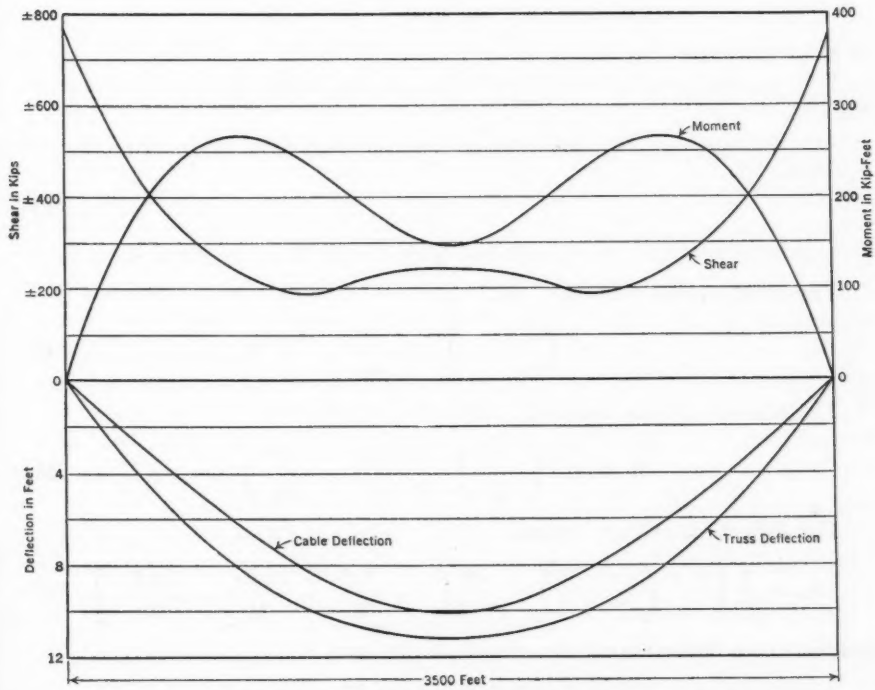


FIG. 15.—GEORGE WASHINGTON BRIDGE: MAXIMUM SHEAR, MOMENT, AND DEFLECTION DIAGRAMS.

George Washington Bridge under various conditions of loading. Ten equations were used in the analysis. The results for the full loading were identically checked by the elastic curve method described herein.

(*3*).—*Approximate Method.*—It can easily be perceived that there is no transfer of wind pressure from the stiffening trusses to the hangers at the ends of trusses since there is no deflection at these points. It is also obvious that the restitution, $r$, will be maximum at the center of the span in case of full wind pressure. For this reason the uniform wind distribution is not representative of the actual conditions. A closer approximation of the true nature

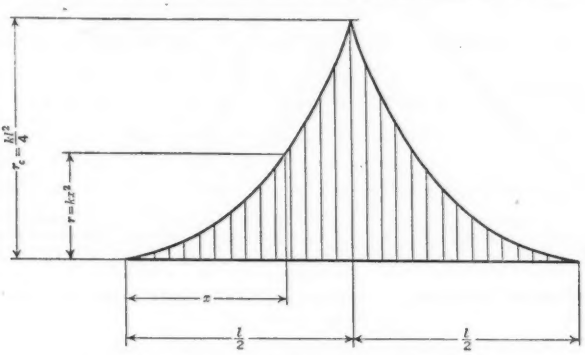


FIG. 16.—PARABOLIC RESTITUTION DIAGRAM.

of the distribution, as suggested by the results of the exact methods, may be had by assuming a parabolic variation of the restitution, $r$, as shown in Fig. 16.

On the basis of this assumption the restitution is then given by the equation:

$$r = kx^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

The pressure sustained by the trusses is expressed then, $w_t - r = w_t - kx^2$.

The shear at any section, $x$, is:

$$Q = w_t \left(\frac{l}{2} - x\right) - \left(\frac{kl^3}{24} - \frac{kx^3}{3}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

The moment at any section, $x$, is:

$$M = w_t\, x \left(\frac{l-x}{2}\right) - \left(\frac{kl^3 x}{24} - \frac{kx^4}{12}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The deflection at any section, $x$, is:

$$\delta_{tx} = \frac{1}{EI}\left[\frac{w_t\, x^3}{3}\left(\frac{x}{12} - \frac{l}{6}\right) + \frac{k\, l^3\, x^3}{144} - \frac{kx^6}{360} + \frac{w_t\, l^3 x}{24} - \frac{3\, k\, l^5 x}{640}\right] \ldots\ldots (47)$$

and the truss deflection at the center of the span is:

$$\delta_t = \frac{5\, w_t\, l^4}{384\, EI} - \frac{7\, kl^6}{4\,608\, EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

By isolating the cable and taking the moments about the center, $H\delta_c = \frac{k\,l^4}{48} - \frac{k\,l^4}{192} + \frac{w_c\,l^2}{8}$; whence,

$$\delta_c = \frac{k\,l^4}{64\,H} + \frac{w_c\,l^2}{8\,H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

By Equation (38) $\Delta\,\delta_x = \frac{h_x}{p_{sx}}\,r$; whence,

$$\Delta\,\delta = \frac{hkl^2}{4\,p_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

Equating the deflections, $\delta_t = \delta_c + \Delta\,\delta$,

$$\frac{5\,w_t\,l^4}{384\,EI} - \frac{7\,k\,l^6}{4\,608\,EI} = \frac{k\,l^4}{64\,H} + \frac{w_c\,l^2}{8\,H} + \frac{h\,k\,l^2}{4\,p_s}$$

and solving for $k$,

$$k = \frac{\frac{5\,w_t}{384\,EI} - \frac{w_c}{8\,H}}{\frac{l^2}{64\,H} + \frac{7\,l^4}{4\,608\,EI} + \frac{h}{4\,p_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (51)$$

With $k$ known, the moments, shears, and deflections can be readily determined by the use of Equations (45) to (47).

The method gives very close results as compared with more exact methods of analysis.

*Determination of $H_w$.*—In all the previous analyses, $H$ has been treated as a known quantity. It should be noted that it is also unknown since it involves the unknown $H_w$. In as much as $H_w$ can be found only after the actual cable loads are obtained, it is necessary first to assume a certain $H_w$ at the beginning of the analysis, to use $H = H_a + H_w$, and, after finding the actual cable loads, to check $H_w$. If the discrepancy between the assumed and the actually computed $H_w$ is substantial, another trial $H_w$ may be used, and the process continued until the desired agreement between the assumed and the computed $H_w$ is reached. The additional horizontal component, $H_w$, does not usually amount to more than 2% of the total $H$. Its omission from the computations is on the safe side in so far as wind stresses are concerned.

*Effect of Tower Deflection.*—In case of appreciable transverse tower deflection the true deflection relation becomes $\delta_{tx} = \delta_{cx} + \Delta\,\delta_x + u$. Equations (39) to (43) should be modified accordingly by adding the constant, $u$, to the right-hand side of these equations. In Equations (23) and (51), this change should be introduced in the derivation.

It is hoped that the foregoing discussion will indicate some preferable variations in the analysis of wind-pressure distribution as presented by the authors.

RALPH A. TUDOR,[6] JUN. AM. SOC. C. E. (by letter).[6a]—Tests made on a model of one of the suspension bridges proposed and studied for crossing the west channel of the Bay between San Francisco and Oakland, Calif., offered a rare opportunity to investigate the method developed by Messrs. Moisseiff and Lienhard to determine lateral deflections and stresses produced in the suspended structure by wind. Heretofore, this particular problem had not been answered satisfactorily, and it is gratifying that by analytical and mechanical methods a close check of results has been obtained.

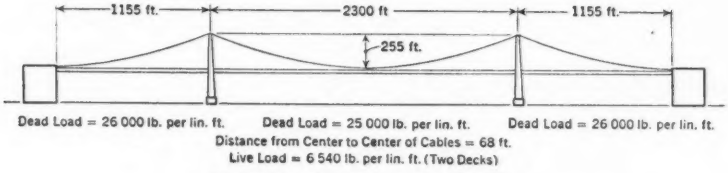
1155 ft.
2300 ft
1155 ft.
255 ft.
Dead Load = 26 000 lb. per lin. ft.
Dead Load = 25 000 lb. per lin. ft.
Dead Load = 26 000 lb. per lin. ft.
Distance from Center to Center of Cables = 68 ft.
Live Load = 6 540 lb. per lin. ft. (Two Decks)

FIG. 17.—ELEVATION OF BRIDGE.

Fig. 17 illustrates the general dimensions of the prototype, which was reproduced with a linear scale reduction of 1:100. Since a complete description of the models and test procedure has been published elsewhere[7] these details are not repeated herewith.

Comparative values of truss reactions and deflections of cables and truss at mid-span were as listed in Table 14.

TABLE 14.—COMPARISON OF TRUSS REACTIONS AND DEFLECTIONS

| Item | Method | Truss Reaction at the Tower, in Kips | | Cable Deflection at the Center Line of the Main Span, in Inches | | Truss Deflection at the Center Line of the Main Span, in Inches | |
|---|---|---|---|---|---|---|---|
| | | Full live load | No live load | Full live load | No live load | Full live load | No live load |
| 1 | Uniform distribution | 657 | 461 | 70 | 50 | 73 | 52 |
| 2 | Elastic distribution. | 858 | 586 | 60* | 49* | 76* | 54* |
| 3 | Model test..... | 989 | 659 | 67* | 48* | 74* | 53* |

* See, also, Figs. 18(*a*) and 18(*b*).

Fig. 18(*a*) clearly shows the extremely close agreement obtained in the deflection of all points along the truss and cable by the elastic distribution method and the model with no live load on the structure. Fig. 18(*b*) shows the comparative values with full live load on the bridge. Fig. 19 illustrates the load distribution curves as computed for the two cases.

Due to the fact that bending moments caused by wind were determined with partial live loads on the model it was difficult to make a direct comparison of values. However, such a comparison was made with live load covering approximately the middle-third of the main span and the following bending moments, in foot-kips, at the center of the span were found: Elastic distribution method, 3 300, and model tests, 3 020.

[6] Asst. Designing Engr., San Francisco-Oakland Bay Bridge, San Francisco, Calif.

[6a] Received by the Secretary April 26, 1932.

[7] Univ. of California *Publication in Engineering*, Vol. 3, No. 2.

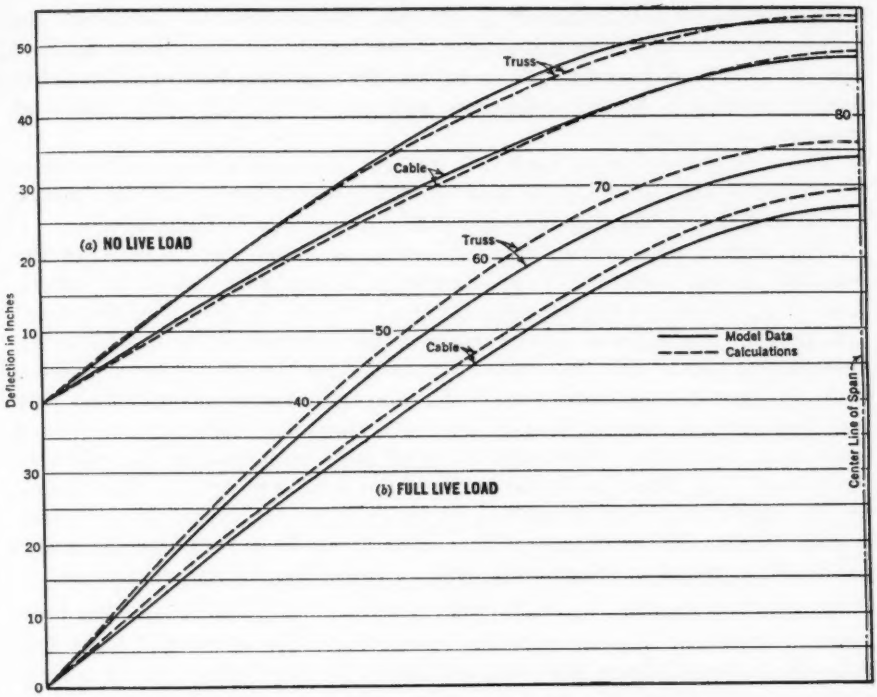


FIG. 18.—LATERAL DEFLECTION OF TRUSS AND CABLE.

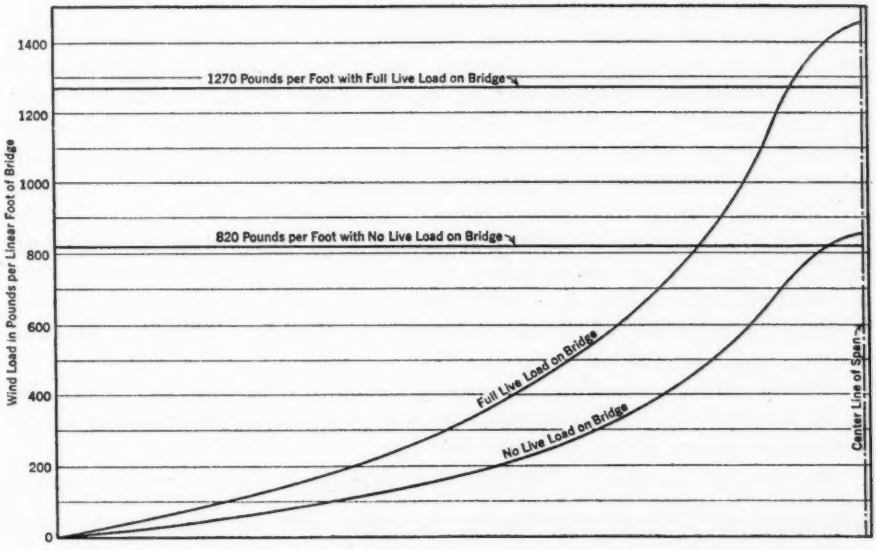
Wind Load in Pounds per Linear Foot of Bridge
1270 Pounds per Foot with Full Live Load on Bridge
820 Pounds per Foot with No Live Load on Bridge
Full Live Load on Bridge
No Live Load on Bridge
Center Line of Span

FIG. 19.—DISTRIBUTION OF WIND LOAD.

*Conclusions.*—As indicated by Messrs. Moisseiff and Lienhard, the uniform distribution method is simple of application, but the results obtained may be somewhat in error. It is sufficiently accurate, however, for determining maximum lateral deflections and for preliminary calculations of moments and shears.

The elastic distribution method is without appreciable error and provides the designer with a reliable means of obtaining bending moments, shears, and deflections of the stiffening truss of a suspension bridge.

The studies upon which this discussion is based clearly indicate the inherent lateral stability of a suspension bridge with adequate towers. Actual deflections are more greatly influenced by the dead load of the structure than by the stiffness of the trusses. The assumption of the following absurdity supports this statement: Consider the bridge discussed herein with all functions unaltered, except that the trusses are constructed so that they have zero moments of inertia about a vertical axis and are unrestrained laterally at the towers. Obviously, all wind must be carried by the cables to the towers. The deflection of the center of the main span truss is then found, by the elastic distribution method, to be only 11.1 ft. when the structure is fully loaded. This compares with a value of 6.2 ft. as found on the model and 6.3 ft. as found by calculation, with a truss of normal lateral stiffness.

F. H. CONSTANT,[8] M. AM. SOC. C. E. (by letter)[8a]—Much thought has been given to stresses in suspension bridges under the action of vertical loads, but those due to lateral forces have received scant attention. However, in narrow and moderately long spans, the latter may easily be considerably greater than those produced by the live load. This paper, in presenting a method, at once correct and simple, for the determination of the wind stresses in the lateral system of a suspension bridge, is very timely.

The practical usefulness of any trial-and-error method increases with the data acquired by repeated applications. With this in mind the writer would like to add another example, that of the Mt. Hope Bridge in Rhode Island, to those already cited by the authors. In Fig. 20 are shown wind chord stresses for this bridge by each of the authors' two methods, for the case of wind acting uniformly over the entire main span.

The data used in the calculations are as follows: $l = 1\,192$; $h_c = 118.57$; $h_t = 124.57$; $w_c = 40$; $w_t = 360$; $H$ (both cables, for combined dead load and live load over the entire center span, summer temperature) $= 9\,588\,000$; $I$ (wind truss) $= 19\,600$; $E = 29\,000\,000$; $P_{sx}$ (combined dead load and live load) $= 6\,800$; width of bridge (centers of stiffening trusses) $= 34$; and hanger length at center $= 6$.

By the uniform distribution method, $z = 241$ lb., leaving 119 lb. per lin. ft. as the uniform load to be taken by the truss. The corresponding chord stresses are shown by the parabola, Fig. 20.

In applying the elastic distribution method, the variation in $I$ was taken into account, $\frac{I_c}{I_x}$ being 1.00 between the center and the quarter-point, and

[8] Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

[8a] Received by the Secretary June 18, 1932.

increasing to 1.30 at the end. Fig. 21 is the load distribution diagram (compare with the authors' examples). The synchronism between the cable and truss deflections at each of the ten sections chosen for the half-span, checked to within 0.10 ft. and generally was much closer. The correspond-

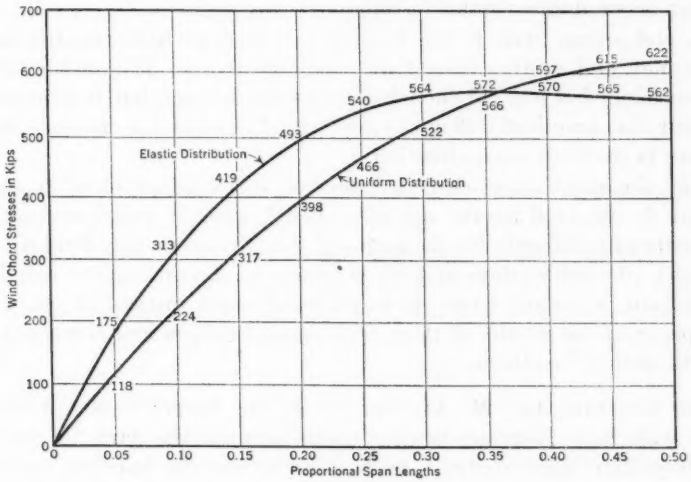


FIG. 20.—WIND CHORD STRESSES BY ELASTIC AND UNIFORM DISTRIBUTION METHODS.

ing chord stresses are shown by the upper curve in Fig. 20. For full wind loading over a symmetrical span the method is simple of application and accurate in results.

The maximum live load chord stresses in the stiffening truss, however,

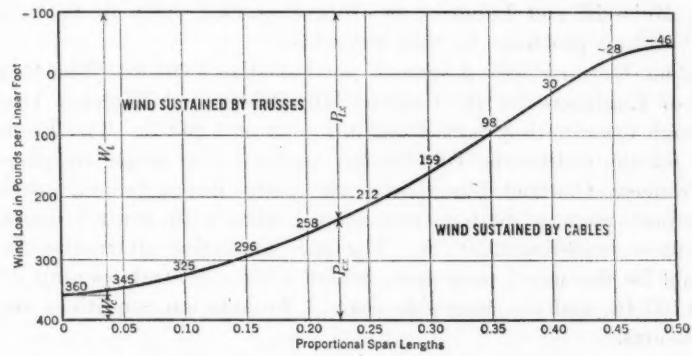


FIG. 21.—WIND DISTRIBUTION DIAGRAM—MT. HOPE BRIDGE—ELASTIC DISTRIBUTION METHOD.

do not arise when the span is fully loaded, but when the live load extends over only a part of the span. This not only changes the value of $H$, but as some of the lateral wind force must be due to the exposed surface of the live load, the actual wind loading on the span at the time the live load moments

are maximum, cannot be uniform over the entire span. In fact, this simple case is scarcely applicable at all, since every case of maximum live load moment is accompanied by a corresponding case of unsymmetrical wind loading; and, similarly, for the maximum diagonal stresses in the lateral system that occur under partial loading.

In the end spans, even if the loading is taken as uniform, the hanger lengths are not, and another case of dissymmetry arises. The authors' elastic distribution method is general enough to cover these cases, but it is important to point out the practical difficulties and the fact that the authors' simple loading case is itself an approximation.

Probably the most satisfactory solution of the wind problem in suspension bridges is obtained by the use of a model, such as was constructed at the University of California for the proposed San Francisco Bay Bridge. With such a model all combinations of both vertical and lateral loading conditions can be tried out in a short time. It would be of much interest in connection with this paper, if the results of those tests could be given and compared with those by the authors' methods.

CHARLES DERLETH, JR.,[9] M. AM. SOC. C. E. (by letter).[9a]—San Francisco Bay offers wide water barriers to easy traffic between the parts or boroughs of its metropolitan community. At San Francisco the barriers are much wider and deeper than at New York. For the past twenty-five years, in the writer's experience, costs of various bridge and tunnel projects have been bitterly debated and adequate truss dimensions disputed. There were those who clung to the belief that width of bridge and depth of truss must bear a direct ratio to length of span; and that weight and cost must bear a direct ratio to some function or power of the span, irrespective of whether the bridge is short or long, and without distinction in type. The contribution by Messrs. Moisseiff and Lienhard is of fundamental value in the solution of bridge building problems in San Francisco.

The Golden Gate requires a span of not less than 4 000 ft. The Consulting Board of Engineers for the Golden Gate Bridge and Highway District, after thorough examination, have fixed this span at 4 200 ft. The Engineering Board for the California Toll Bridge Authority, in preparing plans for the San Francisco-Oakland Bay Bridge *via* Yerba Buena Island, provisionally determined upon a double suspension bridge with central anchorage involving spans exceeding 2 300 ft. The only attractive alternative to this design would be the usual three-span bridge with a central opening of not less than 4 100 ft., and two long side spans. Foundation conditions require these dimensions.

It is evident then that for either the Golden Gate Bridge, or the trans-Bay projects, long spans are mandatory, and width of bridge is of the utmost importance. Messrs. Moisseiff and Lienhard have presented a notable and ingenious argument throwing light upon the capacity of long-span suspension bridges to withstand lateral wind forces.

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[9] Cons. Engr.; and Dean, Coll. of Eng., Univ. of California, Berkeley, Calif.

[9a] Received by the Secretary July 1, 1932.

Great public projects in their pioneering days frequently meet with criticism and obstruction, due to conflict between proposed improvements and the existing order. The writer has no quarrel with those who object, enjoin, or criticize. In the long run a needed improvement receives impetus from such debating, which stimulates thorough study of the questions raised, and leads to discovery and invention in the science of design and to improvement in the art of construction. The paper had its inception in the early debates on the Golden Gate design. The authors were seeking a convincing argument to justify shallow stiffening trusses and slender wind-bracing for a 4 000-ft. span.

Testimony offered at San Francisco in opposition to the early Golden Gate Bridge designs argued that the designs presented were of inadequate width; that they did not have the necessary lateral stability. The proposed width between cables for the Golden Gate Bridge is 90 ft. Engineers of considerable accomplishment argued that, for a 4 000-ft. span, these widths should exceed 100 ft. and, perhaps, approach 200 ft. Necessarily, their estimates of cost greatly exceeded those of the criticized design. Messrs. Moisseiff and Lienhard's analysis completely answers these criticisms and gives to their paper a fundamental practical and public value, in addition to the merit which should be accorded them for an elegant analysis of a difficult problem.

They have clearly shown that the long-span suspension bridge is different in nature from simple trusses, arches, and cantilevers, and even from suspension spans of less length. One might speak of different species in a genus of suspension bridges. With long spans the cable becomes by far the stiffest member in the bridge system. It reduces the importance of the stiffening truss for vertical load distribution and likewise eliminates largely the necessity for heavy transverse wind framing and great width in the plane of the bridge floor. Sometimes, the writer likes to describe the theory of Messrs. Moisseiff and Lienhard as assigning to the floor system the nature of a kite in the wind, with the cable and suspenders acting as the restraining strings and tail. This may be a poetic rather than a mechanical figure of speech, but the analogy is clear. The authors have caused bridge engineers to consider long-span suspension structures from a newer point of view with respect to transverse resistance when compared to the usual treatment for wind forces in a simple bridge truss, an arch, or a cantilever.

Thus, the elastic distribution method is found to show that the cables relieve the stiffening trusses of wind load moments about 15% for an East River Bridge of 1 500-ft. span, while for the 3 500-ft. George Washington Bridge the relief is more than 80 per cent. For the Golden Gate Bridge, with two cables and a 90-ft. width of roadway, the relief is more than 50 per cent. These increments of relief are so substantial that, in long-span work, they silence all arguments for unnecessary floor widths, heavy wind framing, cable cradling, or other devices such as lateral stay cables where topography permits. They also silence claims for greatly increased costs, such as were advanced in the pioneer investigations at the Golden Gate.

The elastic distribution method adds additional arguments for the accuracy of cost analysis and the sufficiency of long-span designs, in that the authors show reduced wind reactions at the tower tops resulting from the application of the elastic distribution method; a reduction of 10% for the 1 850-ft. Detroit span, 20% for the George Washington Bridge, and about the same amount for the contemplated structure at the Golden Gate. This advantage becomes especially valuable for San Francisco Bay bridges when the possibilities of wind and other vibrations are considered. The elastic distribution method shows that the tall steel towers require less wind-bracing and receive less lateral deflection than heretofore assumed.

The application of these methods to the bridge from San Francisco to Oakland has justified narrower widths and double-deck floors; thus substantial savings are effected not only in the superstructure, but also in the foundations, whereas in earlier studies it was argued that for this enterprise long spans would demand one floor instead of two and widths exceeding 100 ft. instead of the present contemplated width of 66 ft. between cables and stiffening trusses.

It is gratifying to know that the models studied at the University of California[10] for proposed San Francisco bridges have checked closely the authors' theory.

A. A. EREMIN,[11] ASSOC. M. AM. SOC. C. E. (by letter).[11a]—In this paper the authors have shown some simplified methods of distributing lateral forces between the cables and the trusses.

In the uniform distribution method it was assumed that the transferred wind load from trusses to cables is distributed uniformly over the entire span length. However, the actual distribution is similar to that determined by the authors in the elastic distribution method (Fig. 2(*g*)).

A slight modification of the authors' Equation (5) is required to vary the assumption of wind-load distribution. It is also of interest to find an approximate expression for the effect of tower deflection on distribution of lateral forces. Fig. 22(*a*) shows the transferred wind load in the form of a parabola with the maximum transferred load, $z$, at mid-span. The cross-section in Fig. 22 shows deflections of the towers, the cables, and the trusses at mid-span.

Using the authors' notation, the horizontal deflection of the cables at mid-span for a load that varies as shown in Fig. 22(*a*), is:

$$\delta_c = \frac{l^2\,(1.2\,w_c + z)}{9.6\,H} \qquad \text{.....(52)}$$

and deflection of the trusses is:

$$\delta_t = \frac{l^4\,(1.2\,w_t - z)}{92.5\,E\,I} \qquad \text{.....(53)}$$

[10] Univ. of California *Publication in Engineering*, Vol. 3, No. 2.

[11] Asst. Designing Engr., Bridge Dept., State Highway Comm., Sacramento, Calif.

[11a] Received by the Secretary July 5, 1932.

From the cross-section, Fig. 22, the relation follows that:

$$\frac{h_t}{h_c} = \frac{\delta_t - \delta_a}{\delta_c} \quad \text{.....(54)}$$

in which, $\delta_a$ is the deflection of the towers at the top. Combining Equations (52), (53), and (54), the transferred wind load, $z$, is:

$$z = \frac{11.5\,H\,h_c\,l^4\,w_t - 111.0\,E\,I\,h_t\,l^2\,w_c - 890.0\,E\,I\,H\,\delta_a\,h_c}{92.5\,E\,I\,h_t\,l^2 + 9.6\,H\,h_c\,l^4}$$

or,

$$z = K\left(\frac{w_t}{1+\frac{b}{a}} - \frac{w_c}{1+\frac{a}{b}} - \frac{\frac{c}{b}}{1+\frac{a}{b}}\right) \quad \text{.....(55)}$$

in which, $K = 1.2$, and $c = 77.5\,E\,I\,H\,\delta_a\,h_c$. Values of the other factors are the same as those in Equation (5).

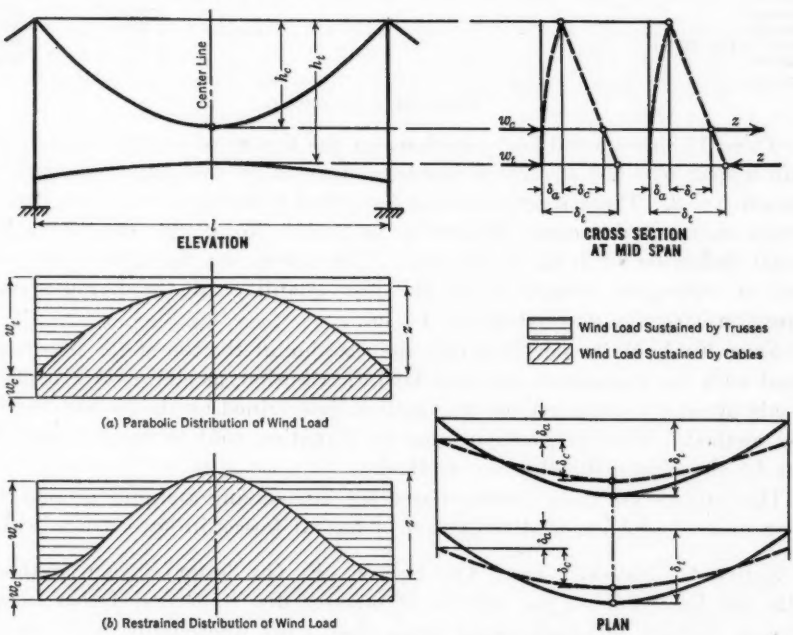


FIG. 22.—DISTRIBUTION OF WIND LOAD.

The last quantity in the parenthesis of Equation (55) expresses the effect of tower deflection on the transferred wind load. The horizontal deflection of the towers at the top, $\delta_a$, may be determined by the successive approximations.

In a long-span suspension bridge the ends of the stiffening trusses are restrained, due to the action of their own weight. Therefore, the distribution of the lateral forces will be similar to that shown in Fig. 22(*b*), in

which the transferred wind load is in the form of a curve, with the maximum ordinate, $z$, at mid-span. An approximate algebraic expression of this curve may be assumed, as follows:

$$y = \frac{z\,(4\,x^2 - l^2)^2}{l^4} \qquad (56)$$

Thus, at mid-span, $x = 0$ and $y = z$.

The transferred wind load, $z$, Fig. 22($b$), may be determined by Equation (55), in which case, $K = 1.35$ must be used. It is interesting to compare the effect of distributing wind load according to the parabola in Fig. 22($a$), a curve, such as Fig. 22($b$), and the distribution methods developed by the authors.

TABLE 15.—COMPARISON OF DISTRIBUTION METHODS

| Distribution of wind load | Wind Reaction on One Tower, in Kips | |
|---|---|---|
| | Top | Truss level |
| Uniform * | 2 060 | 540 |
| Parabolic | 1 700 | 900 |
| Shown in Fig. 22 (b) | 1 565 | 1 035 |
| Elastic* | 1 527 | 1 073 |

* Computed by the authors.

Table 15 shows wind-load reactions on the towers of a suspension bridge, with a span of 4 000 ft., which has been denoted by the authors as proposed Design No. 2. These reactions were computed with the assumption that the towers were not deflected. Referring to Design No. 2, the effect of a horizontal deflection of 1 ft. at the top of the towers, on the transferred wind load at mid-span, computed by the last quantity in the parenthesis of Equation (55), is approximately 14 lb.

From Table 15 it is evident that the reaction at the top of the tower computed with the assumption of wind load distribution, as shown in Fig. 22($b$), is only about 4% greater than the reaction determined by the elastic distribution method. However, computation by Equation (55) is much easier than that by the elastic distribution method.

The authors are to be congratulated for introducing a simple presentation of an evolved problem of distribution of lateral forces in suspension bridges.

ELMER C. OSGOOD,[12] JUN. AM. SOC. C. E. (by letter).[12a]—In textbooks, little can be found on the subject of stresses due to lateral forces in suspension bridges, although many texts deal quite thoroughly with stresses caused by vertical forces. Furthermore, the validity of some of the assumptions made in the few analyses that can be found is questionable. Consequently, the writer has made an investigation:[13] (1) To determine the importance of stresses due to lateral forces; (2) to find a method by which these stresses might be obtained with an exactness consistent with their

[12] Greenfield, Mass.

[12a] Received by the Secretary July 27, 1932.

[13] Presented as a thesis to Rensselaer Polytechnic Institute in partial fulfillment of the requirements for the degree of Doctor of Engineering in Civil Engineering.

importance; (3) to determine if, and for what cases, approximate methods could be used; and (4) to discover what assumptions are valid for a sufficiently exact general solution.

Some of the conclusions were determined for the writer's benefit and are no doubt well known to those who have designed suspension bridges, and, therefore, will not be mentioned. Many of the others have been presented by the authors and, since they agree, need no further comment. Some of the results, which it is believed are not generally known, or which are not generally available, are presented herein.

The nature of the problem was investigated thoroughly, and a relation between the component parts of the suspended system was set up similar to that used by the authors in the elastic distribution method. The writer then investigated two approximate theories: (1) That presented[14] by D. B. Steinman, M. Am. Soc. C. E., and (2) that presented[15] by Johnson, Bryan, and Turneaure.

Sufficient reason for accepting some of the assumptions made in the approximate theories could not be found; therefore, a more exact solution was sought. The attempts resulted in failure in so far as reaching the objective was concerned, but in each case the reasons why the method should fail, were found. Evidently, no entirely analytical exact solution has yet been devised, although efforts no doubt have been made to do so. It seems quite probable that more attempts will be made to this end in the future, because present methods are long and tedious. It would be well, then, that any logical attempts that have failed, be made generally known, since this would prevent not only possible duplication of useless effort when the reason for failure could be shown, but might also point the way to a different solution for some one else.

From the relation set up between the parts of the suspended system, using only those assumptions which the authors used in the elastic distribution method, the writer was able to derive the equation:

$$E\,I\,\frac{d^2\,\delta_{tx}}{dx^2} - H\,\delta_{tx} + \frac{H\left[h_t - \frac{4\,h_c\,x}{l^2}\,(l-x)\right]\left[w_t - E\,I\,\frac{d^4\,\delta_{tx}}{d\,x^4}\right]}{p_{sx} - u_x} = (w_t + w_c)\,\frac{x^2}{2} + K_1\,x + K_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

The authors' notation has been used as far as possible. The factor, $u_x$, is the variable load per unit length of span sustained by the beam action of the vertical truss system at a distance, $x$, from one end; $K_1$ and $K_2$ are constants to be determined from the conditions of loading. The expression for $u_x$ may be found in terms of $x$ by any of the common formulas for vertical truss analysis. When no vertical trusses are used, $u_x = 0$.

Therefore, the equation can be made to contain, other than constants, only the independent variable, $x$, and its dependent variable, $\delta_{tx}$. Although

[14] "Suspension Bridges and Cantilevers", by D. B. Steinman, D. Van Nostrand Co., N. Y., 1913, p. 75.

[15] "Modern Framed Structures", by J. B. Johnson, C. W. Bryan, and F. E. Turneaure, Pt. II, Tenth Edition, John Wiley & Sons, N. Y., 1929, p. 273.

the writer did not know of, and could not find, any method of direct integration, mathematically a solution is possible by the use of an infinite series. An infinite power series,

$$\delta_{tx} = A + Bx + Cx^2 + Dx^3 + \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

was assumed. The necessary derivatives of this equation may be found and substituted in Equation (57). The sums of the coefficients of like powers of $x$ may then be equated to zero and the values of the constants, $A$, $B$, $C$, etc., determined. A sufficient number of terms must be used in the series to insure that it be convergent.

By several trials in characteristic problems, it was found that, for the number of terms used in the series (13 were used in some cases), the equation when solved for large values of $x$ did not even begin to converge. It might be argued that the equation could be used for finding values for the part of the span in which $x$ is small, but it may be proved that, because one of the necessary conditions which must be introduced is that $\delta_{tx} = 0$ when $x = l$, the equation does not result in correct values for any part of the span unless it is convergent for all points. The only inference that could be drawn, therefore, was that the infinite power series is a form unsuited to express the deflection curve of the horizontal trusses.

It was found that another reason for the failure of this method was the necessity of expanding $u_x$ into an infinite power series to express it in terms of $x$ and constants. This series also required too large a number of terms to make it convergent.

An attempt was made to solve the equation by assuming an infinite trigonometric series similar to that used by Mr. S. Timoshenko,[16] in his analysis of vertical trusses. The equation was shown to be quickly convergent for the case that he considered, and it is reasonable to suppose that a similar one for horizontal deflection should be of like character. No trouble was encountered in a consideration of the internal work; but a treatment of the external work led to an expression which was so complicated (since it necessitated the use of the derivatives of different types of functions), that the method failed.

The writer had reached this stage, when, through the courtesy of G. M. Rapp, Jun. Am. Soc. C. E., he learned of the elastic distribution method. He decided then to discontinue further investigation for a method that was entirely analytical.

Comparisons were made on the Delaware River Bridge, the Detroit River Bridge, and the George Washington Bridge, using the two approximate theories previously mentioned and the elastic distribution method. The uniform distribution method described by the authors was not known to the writer. Mr. Steinman's method, however, is somewhat similar, although, in the longer spans it results in somewhat greater discrepancies than the authors' approximate method. It was found, nevertheless, that, for the types of bridges for which the approximate methods investigated by the writer

[16] "The Stiffness of Suspension Bridges", by S. Timoshenko, *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 377.

were obviously intended (that is, for those in which the span length, wind load, and truss stiffness were such as to make impossible large cable restraint), these theories gave sufficiently accurate results to be used. Mr. Steinman's method is very simple in its application, and has a tendency toward safe values, especially in the longer spans. The discrepancies in the results of the method of Johnson, Bryan, and Turneaure are of about the same order of magnitude, but tend in the opposite direction. The moment curves of neither method reverse near the center in long spans as does the true moment curve. For the George Washington Bridge, because of these characteristics, the Steinman method gave nearly true values for the stresses near the ends of the span; whereas, the other method resulted in values very close to the true ones at the center.

In the derivation of both the approximate methods which were investigated by the writer (as is also the case at mid-span in the authors' uniform distribution method), the assumption was made that the hangers, prolonged at any point of the span, would intersect the line joining the tower tops. To determine how true such an assumption was, the writer plotted the hanger directions at several points for each bridge. It was found that the variations in the direction of the hangers from the assumed direction were greater for those spans for which the discrepancies between the results of the approximate and the elastic distribution method were greater, and much in the same ratio. This indicated that the assumption made in the approximate methods has a major effect upon the accuracy of the method and is not valid for a solution covering all cases.

The writer investigated the effect of not considering the additional stress in the cable caused by lateral forces when computing the truss stresses induced by those forces. For the George Washington Bridge, under wind over the entire span, this additional stress was found to be only a small part of 1% of the total cable stress, and, therefore, was of negligible importance. Since in this bridge there is a large cable restraint on the truss, it is evident that, in others, which are laterally stiffer and shorter, the percentage is much less.

The importance of the stresses induced in the vertical trusses as a result of the cable and hanger inclination caused by the horizontal deflection, with consequent uplift, was also determined. For the George Washington Bridge under wind over the entire span, the vertical rise at the center caused by the horizontal displacement was found to be a small fraction of a foot. The stresses induced in the vertical trusses by such a deflection are of negligible importance. As in the preceding case, in bridges which are laterally stiffer and shorter, these stresses would be even less.

An investigation was made for the Delaware River Bridge and the Detroit River Bridge to determine how closely the position of loading for maximum live load and temperature stress at any point compared with that for maximum total stress (including, also, stress due to wind on truss and moving load). Using increments in the loaded length of one panel, it was found that for most points the positions for the two cases were the same, whereas for a few points the difference was only one panel length. The

greatest difference in the total stresses for the two cases was found to be less than 0.2% of the maximum total stress.

The effect of temperature upon wind stresses was determined for the Delaware River Bridge. It was found that the maximum difference between the stresses at extreme and mean temperatures was only about 0.5% of the stress at mean temperature.

Wind stresses in the chords of the side spans of the Delaware River Bridge were investigated. These stresses were found to be only about 5% of those induced by vertical loading. Since in long suspension bridges the tendency is toward relatively short side spans, it is evident that in the side spans of these bridges, lateral stresses are of even less importance. Furthermore, since the cable causes very little restraint upon the side spans, simple beam methods are sufficiently exact to determine these stresses.

The writer is glad to find that the conclusions he had drawn regarding the span lengths for which approximate methods could be used are practically the same as those of the authors. In no way do any of his findings disagree with the authors' proposals. The purpose of his discussion is rather to illustrate some of the difficulties of obtaining a simple but exact solution, and to clear up a few points of doubt which might occur to the student, if not to the engineer familiar with suspension bridge design.

CHARLES M. SPOFFORD,[17] M. AM. SOC. C. E., AND JOHN B. WILBUR,[18] ASSOC. M. AM. SOC. C. E. (by letter).[18a]—This paper furnishes a long needed treatment, in English, of the subject of wind stresses in suspension bridges, and the authors are to be commended for presenting it to the Society.

The authors' elastic distribution method is essentially a method of trial. Like other trial methods, it involves considerable labor, and any process that will shorten the calculations is helpful. The writers consider that the following method of determining approximately the forces of restitution may be used to advantage in applying the elastic distribution method given by Messrs. Moisseiff and Lienhard, thus reducing the number of trials necessary.

This method is based upon the excellent treatment of the subject given by E. Pigeaud,[19] supplemented by certain charts and tables prepared by the writers and A. F. Ghiglione, Jun. Am. Soc. C. E.

*Notation.*—In addition to the notation adopted by the authors, let,

$a$ = one-half the length of the central span, in feet;
$ma$ = the distance from center of span to any point under consideration;
$K$ = a constant, depending upon variation in transfer of wind stresses along the span;
$k$ = a constant depending upon magnitude of the restitution force resulting from the slope of the hangers due to horizontal deflection of the truss;

[17] Hayward Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge; Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

[18] Instr., Dept. of Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

[18a] Received by the Secretary July 20, 1932.

[19] *Le Genie Civil*, Vol. 87, July, 1925.

$I$ = average value of moment of inertia of lateral truss (an approximate average value of $I$ based upon the moment of inertia of chords alone may be used, or a more exact value may be computed by determining the center deflection of the truss under a full uniform load by one of the usual methods, equating it to the center deflection of a beam under uniform load and solving for $I$);

$M_r$ = moment on truss due to restitution;

$M_w$ = moment on truss due to wind load, assuming truss as end supported;

$V_r$ = end shear on wind truss due to restitution;

$V_w$ = end shear on wind truss due to wind load, assuming truss as end supported;

$\delta_{cw}$ = total lateral deflection of the cable at any section, due to wind, assuming no lateral force in hangers;

$\delta_{tw}$ = total lateral deflection of the truss at any section, due to wind, assuming no lateral force in hangers;

$\delta_{cr}$ = total lateral deflection of the cable at any section, due to the lateral forces in the hangers;

$\delta_{tr}$ = total lateral deflection of the truss at any section, due to the lateral forces in the hangers;

$G = \dfrac{k}{\Delta\,\delta}$, with $\Delta\delta$ measured at the quarter-point of the main span;

$$F_m = \frac{1 - m^2}{1 + K m^2};$$

$$F_{1(m)} = \int_0^m \frac{1 - m^2}{1 + K m^2}\, dm;$$

$$F_{2(m)} = \int_m^1 F_{1(m)}\, dm;$$

$$F_{3(m)} = \int_0^m F_{2(m)}\, dm; \text{ and,}$$

$$F_{4(m)} = \int_m^1 F_{3(m)}\, dm.$$

From Equation (11), which is an exact expression:

$$z_x = \frac{p_{sx}\,\Delta\,\delta_x}{h_x} \qquad (59)$$

A study of the distribution of the wind forces to the various hangers of the actual bridge shows that the following equation suggested by Pigeaud may be used to determine $z_x$ with a considerable degree of precision for the case of a uniform load over the entire span:

$$z_x = p_{sx}\frac{k\,(1 - m^2)}{1 + K m^2} = p_{sx}\,k\,F_{(m)} \qquad (60)$$

Equations (59) and (60) give,

$$K = \frac{1}{m^2}\left[\frac{k h_x\,(1 - m^2)}{\Delta\,\delta_x} - 1\right] \qquad (61)$$

Let $G = \frac{k}{\Delta\delta_{(0.5)}}$ (in which, the subscript in the denominator indicates that $\Delta\delta$ is to be measured at the quarter-point of the span; that is, where $m = 0.5$), and plot a curve for $G$ against span length. The result is shown in Fig. 23, which is based upon the solution of a number of important suspension bridges.

Substituting the value of $G$ in Equation (61) and letting $m = 0.5$, gives the following equation for $K$:

$$K = 3\ Gh_{(0.5)} - 4 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (62)$$

from which the value of $K$ may be obtained by using the value of $G$ from Fig. 23 and the value of $h$ at the quarter-point of the span. For example,

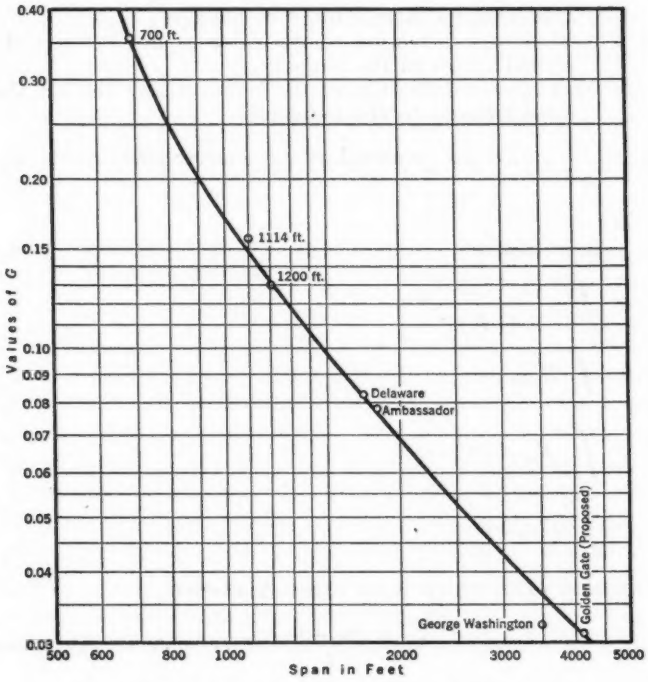


FIG. 23.—CURVE OF $G$ FOR USE IN EQUATION (62).

consider a bridge with a span of 1 750 ft., with a length, $h_{0.5}$, of 58 ft. at the quarter-point. From Fig. 23, $G = 0.0805$ and from Equation (62), $K = 10$.

It is to be noted that, since $Gh$ is a ratio, values of $G$ are in units of $\frac{1}{\text{distance.}}$

With $K$ known, the value of the constant, $k$, may readily be determined, as follows: It is evident that,

$$\delta_{cw(0)} + \delta_{cr(0)} + \Delta\,\delta_{(0)} + \delta_{tr(0)} = \delta_{tw(0)} \quad \ldots\ldots\ldots\ldots\ldots (63)$$

It is also apparent that $\delta_{tw(0)} = \frac{5}{384}\frac{w_t L^4}{E I}$, and $\delta_{cw(0)} = \frac{w_c L^2}{8 H}$, in which, $H$ is the sum of horizontal components in both cables.

From Mechanics,

$$\frac{d^2 \delta_{tr}}{dx^2} = \frac{M}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (64)$$

Furthermore, the first differential of the moment on a beam divided by $EI$ equals the shear divided by $EI$, and the second differential of the moment divided by $EI$ equals the load per foot divided by $EI$; hence,

$$\frac{d^2 M}{dx^2} = E I \frac{d^4 \delta_{tr}}{dx^4} = z_x = k\, p_{sx} \left(\frac{1 - m^2}{1 + km^2}\right) = k p_{sx} F_{(m)} \ldots\ldots\ldots (65)$$

from which, the shear due to restitution is equal to:

$$\frac{E I d^3 \delta_{tr}}{dx^3} = k p_{sx} \int_0^x F_m dx = k p_{sx}\, a F_{1(m)} = \frac{k p_{sx} L}{2} F_{1(m)} \ldots\ldots (66)$$

Therefore, $\frac{E I d^2 \delta_{tr}}{dx^2} = \frac{k p_{sx} L^2 F_{2(m)}}{4}$; and $\frac{E I d \delta_{tr}}{dx} = k p_{sx} \frac{L^3}{8} F_{3(m)}$; and,

$$\delta_{tr} = \frac{k p_{sx} L^4 F_{4(m)}}{16 E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (67)$$

Equation (67) gives the deflection of any point on the truss due to restitution only.

The deflection of the cable at any point $= \frac{M}{H}$ (see Equation (9), in which, $M$ equals the moment at the section of an end-supported beam at the same point, due to a similar loading; but from the foregoing,

$$M = \frac{E I d^2 \delta_{tr}}{dx^2} = \frac{k p_{sx} L^2 F_{2(m)}}{4} \quad \ldots\ldots\ldots\ldots\ldots\ldots (68)$$

therefore,

$$\delta_{cr} = \frac{k p_{sx} L^2 F_{2(m)}}{4 H}$$

From Equation (59),

$$\Delta \delta_x = \frac{h_x}{p_{sx}} z_x = \frac{h_x}{p_{sx}} \left(\frac{p_{sx}\, k\, (1 - m^2)}{1 + Km^2}\right)$$

which, when $m = 0$, gives the expression, $\Delta \delta_{(0)} = h_{(0)} k$.

Substituting the foregoing values of $\delta_{tw}$, $\delta_{cw}$, $\delta_{tr}$, $\delta_{cr}$, and $\Delta \delta_0$ in Equation (63) and solving for $k$ gives,

$$k = \frac{\frac{5}{384}\frac{w_t L^4}{EI} - \frac{w_c L^2}{8 H}}{\frac{p_{sx} L^4}{16 E I} F_{4(0)} + \frac{p_{sx} L^2}{4 H} F_{2(0)} + h_0} \quad \ldots\ldots\ldots\ldots\ldots (69)$$

Values of $F_{2(0)}$ and $F_{4(0)}$ may be computed for various values of $K$ and tabulated for repeated use. Thus, $k$ may be readily determined for any given bridge. With $K$ and $k$ known, the value of $z_x$ at any point of the truss may be easily obtained from Equation (60).

The values of $z_x$ thus obtained may be used as trial values in the elastic distribution method and should reduce materially the number of trials necessary to give a satisfactory solution of the problem.

The application of this method to the determination of trial values of $z_x$ for use in the elastic distribution method, as applied to the 4 000-ft. span solved by Messrs. Moisseiff and Lienhard, is as follows: From Fig. 23, $G = 0.0318$; whence, by Equation (62): $K = 3(0.0318)\ (141.2) - 4 = 9.47$; $\frac{5}{384}\frac{w_t L^4}{EI} = 81.595$; $\frac{w_c L^2}{8\,H} = 2.727$; $\frac{p_{sx}\ L^4}{16\,EI} = 7492.550$; and $\frac{p_{sx}\ L^2}{4\,H} = 800.000$. From the pre-arranged table, $F_{2(0)} = 0.261$; $F_{4(0)} = 0.100$; and hence, by Equation (69),

$$k = \frac{81.595 - 2.727}{7492.550\ (0.100) + 800\ (0.261) + 10.00} = 0.0815$$

Table 16 gives the trial values of $z_x$, as computed by the writers' method, as compared with the final values determined by the authors' elastic distribution method.

TABLE 16.—COMPARISON OF TRIAL VALUES OF $z_x$ (IN POUNDS PER FOOT), AS COMPUTED BY THE WRITERS' METHOD, COMPARED WITH THE FINAL VALUES DETERMINED BY MESSRS. MOISSEIFF AND LIENHARD BY THE ELASTIC DISTRIBUTION METHOD.

| $m$ | Trial values by writers' method | Final values by elastic distribution method | $m$ | Trial values by writers' method | Final values by elastic distribution method |
|---|---|---|---|---|---|
| 0.0 | 1 793 | 2 000 | 0.6 | 260 | 290 |
| 0.1 | 1 623 | 1 650 | 0.7 | 162 | 180 |
| 0.2 | 1 248 | 1 115 | 0.8 | 92 | 105 |
| 0.3 | 881 | 790 | 0.9 | 39 | 45 |
| 0.4 | 599 | 550 | 1.0 | 0 | 0 |
| 0.5 | 400 | 410 | ..... | ..... | ..... |

The value of the truss moments due to lateral forces may be computed directly without the determination of $z_x$, by using curves of $F_{2(m)}$ plotted against $m$ for various values of $K$.

From Equation (68) the moment in the truss due to restitution equals $M_r = \frac{k p_{sx} L^2}{4} F_{2(m)}$. The moment, $M_w$, on the truss, due to wind force, corresponds to the moment on an end-supported beam; hence, the actual moment carried by the truss equals $M_w - M_r$.

If the moments are found directly for the same 4 000-ft. span, the results shown in Table 17 are obtained by utilizing the aforementioned set of curves for $F_{2(m)}$. For purposes of comparison, the values obtained by the elastic distribution method are also inserted in this table. The results show the maximum discrepancy in the moment to occur where $m = 0.2$. The discrepancy at this point amounts to $\frac{18}{190} = 3.7\%$; hence, this method is

sufficiently precise without further investigation to permit the making of a preliminary design, which can then be tested by the elastic distribution method, using the moment curve thus determined for the first trial.

TABLE 17.—COMPARISON OF RESULTANT MOMENTS BY TWO METHODS, IN THOUSANDS OF FOOT-KIPS.

| $m$ | $F_{2(m)}$ | $M_r$ | $M_w$ | Writers' method (approximate) | Elastic distribution method |
|---|---|---|---|---|---|
| 0.0 | 0.261 | 1 872 | 2 300 | 428 | 436 |
| 0.1 | 0.255 | 1 829 | 2 277 | 448 | 453 |
| 0.2 | 0.242 | 1 736 | 2 208 | 472 | 490 |
| 0.3 | 0.221 | 1 585 | 2 093 | 508 | 525 |
| 0.4 | 0.195 | 1 399 | 1 932 | 533 | 547 |
| 0.5 | 0.166 | 1 191 | 1 725 | 534 | 544 |
| 0.6 | 0.134 | 961 | 1 472 | 511 | 511 |
| 0.7 | 0.101 | 725 | 1 173 | 448 | 445 |
| 0.8 | 0.068 | 488 | 828 | 340 | 339 |
| 0.9 | 0.034 | 244 | 437 | 193 | 192 |
| 1.0 | 0 | 0 | 0 | 0 | 0 |

The checking of $\Delta\,\delta_x$ as computed from the various deflections at any point against the value computed from the assumed $z_x$ by the relation, $\Delta\,\delta_x = \frac{h_x}{p_{sx}} z_x$, may be conveniently accomplished by the use of a set of curves giving values of $F_{4(m)}$, plotted against $m$ for various values of $K$. This amounts to checking the value of $K$ taken from Fig. 23.

The determination of the shearing forces may be accomplished as follows: By Equation (66), choosing $F_{1(1.0)}$ from predetermined tabular values, $V_r = 0.0815(22)\ (2\,000)\ (0.345) = 1\,239$ kips, and $V_w = 2\,000(1.150) = 2\,300$ kips. Therefore, the actual end shear equals $2\,300 - 1\,239 = 1\,061$ kips.

In the design of diagonals of the lateral truss, it is necessary to compute the maximum shears at points along the span due to partial wind loads. While the methods outlined by Messrs. Moisseiff and Lienhard are applicable to this condition of loading, the process of determining by trial the distribution of wind transfer is more difficult than for full wind load, since the curve of $z_x$ is no longer symmetrical. An approximate method of obtaining these maximum moments as developed[20] by Mr. W. C. Bodycomb, Jr., may be summarized by the following three steps.

1.—Find the variation and magnitude of the force of restitution for:

(*a*) Wind load over entire span;

(*b*) Wind load from span center to right end.

2.—From the solutions under Step (1), determine the following quantities:

$V_{LRC}$ = left end shear due to restitution with load extending from span center to right end.

$V_{LRE}$ = left end shear due to force of restitution when span is fully loaded.

$\Delta R_c$ = total force of restitution from left end to the center, with load extending from span center to right end.

[20] In a thesis submitted to Massachusetts Institute of Technology in partial fulfillment of the requirement for the degree of Master of Science.

3.—Insert these quantities in the empirical equation:

$$V_{max.} = \left[\frac{w_t L}{8} + \frac{3}{8} V_{LRE} + \Delta R_c\right] m^2 + \left[\frac{w_t L}{4} + V_{LRC} - \frac{11}{8} V_{LRE} - 2\,\Delta R_c\right] m + \left[\frac{w_t L}{8} - V_{LRC} + \Delta R_c\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (70)$$

Substitution of $V_{LRC}$, $V_{LRE}$, and $\Delta R_c$ in Equation (70) reduces it to the form,

$$V_{max.} = Am^2 + Bm + C \quad \ldots\ldots\ldots\ldots\ldots\ldots (71)$$

in which, $A$, $B$, and $C$ are constants for a given bridge. The maximum shear for any value of $m$ is easily computed from Equation (71). As is the case with most involved investigations of indeterminate structures, the time required for computations has made it impossible to compare exact solutions for a large number of bridges with the values given by this empirical curve. For such bridges as have been investigated, however, excellent results were obtained for spans up to 2 000 ft., and reasonably close values for greater spans. Equation (71) should be valuable for preliminary design.

A graphical solution for the various deflections is straightforward, and has been found particularly advantageous for partial wind loads. In connection with the graphical solution one must assume both the type of variation and the magnitude of the $z_x$ curve. The following method will eliminate the necessity of assuming the magnitude correctly:

1.—By the methods of graphical statics construct:

(*a*) A curve of $\delta_{tw}$.

(*b*) A curve of $\delta_{cw}$.

2.—Assume a curve of $z_x$.

3.—On the basis of the assumed curve of $z_x$, construct curves for $\delta_{tr}$ and $\delta_{cr}$ (by the methods of graphical statics) and compute $\Delta\,\delta_x$ at points along the span, by the relation, $\Delta\,\delta_x = h_x \dfrac{z_x}{p_{sx}}$.

4.—At any convenient point (span center in case of full loading), determine the value of $\phi$ by,

$$\phi = \frac{\delta_{tw} - \delta_{cw}}{\delta_{tr} + \delta_{cr} + \Delta\,\delta_x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (72)$$

All the deflections to be used in Equation (72) are measured at the point under consideration.

5.—Multiply all values of $\delta_{tr}$, $\delta_{cr}$, and $\Delta\,\delta_x$, from Step (3) by $\phi$. At the point where $\phi$ is determined, Equation (63) will now hold. Whether or not it holds at other points depends on the type of variation of the $z_x$ curve assumed. If Equation (63) holds at all points, the true values of $z_x$ equal the assumed values multiplied by $\phi$.

Factors not usually taken into consideration in computing wind transfer include tower movement perpendicular to span length, and increase in cable stress due to direct wind load. Either of these factors may be taken into

account by a series of successive solutions, although the latter factor is easily shown to be so small as to be usually negligible. The question may be raised as to what portion, if any, of the live load should be considered as acting in computing restitution forces. Assuming the live load to extend over too great a length of roadway gives too large values for $H$ and $p_{sx}$, the effect of which is to increase $z_x$ and decrease the moments and shears on the lateral truss, which is on the unsafe side as far as the lateral truss stresses are concerned. Considerable uncertainty also exists regarding proper wind loads, time effect, inertia effect, and stiffening effect to the lateral system provided by the roadway.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# FORESTS AND STREAM FLOW

## Discussion

BY C. G. BATES, ESQ.

C. G. BATES,[10] ESQ. (by letter).[10a]—Since the publication in 1928 of the report[11] of the Wagon Wheel Gap stream-flow experiment, the writer has lived in momentary expectation of just such inferences as Messrs. Hoyt and Troxell have made from it. Boldly stated, the authors' conclusion, based on records from two pairs of Western water-sheds, is that vegetation on a water-shed is a detriment to the yield of water. Their revolutionary scheme, brought to its logical conclusion, is to obtain a higher yield of water by means of the simple expedient of removing the vegetation from the water-shed, and with that, eventually, the soil as well. The tone of the paper makes it appear to be directed against foresters who see in the forest cover a beneficial effect.

The writer does not propose to quarrel or quibble over the facts from the water-shed under discussion. By using a different method of analyzing the data obtained from the Wagon Wheel Gap experiment, the authors have changed the run-off percentages by a few tenths of 1 per cent. The stream-flow records in Southern California were obtained without adequate precipitation records for the two water-sheds. These are minor points and are immaterial to the main discussion. On the whole, the authors have stated the facts with careful veracity.

However, they did not need to go to all the trouble of trying to prove that forest cover—or for that matter any vegetative cover—is a source of loss of water to the water-shed. Foresters were the first to demonstrate that forests use water in large quantities, possibly in greater quantities than any other form of vegetation, except deep-rooted perennial crops, such as alfalfa. Foresters, again and again, have pointed out the probability that a water-shed, bare of vegetation, is capable of yielding a larger total quantity of water than the same water-shed if covered with a forest, and the denser and

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and published in August, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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[10a] Received by the Secretary July 2, 1932.

[11] *Monthly Weather Review, Supplement 30.*

more luxuriant the vegetation the greater the loss of water to the water-shed. This is a biological phenomenon no longer in need of proof.

What foresters have claimed and what universal observation since time immemorial has proved is that a forest cover under certain conditions tends to equalize the run-off throughout the year, to increase the storage capacity of the water-shed (especially where the soils are thin or heavy and impervious), and to reduce soil erosion. Foresters have further shown that the beneficial effect of the forest cover varies with the climate, the character of precipitation, and, above all, the character of the soil.

Criticism of this paper is not so much against the authors' conclusions, as they relate to the specific Western water-sheds, but against their attempt to generalize from these water-sheds—which are extreme in character of soil and climate—as to the protective value of the forest under all conditions.

The authors have omitted many facts that are vital to any logical and sane discussion of the subject of forests and stream flow, of "water-shed protection," and of the general utilitarian value of forested *versus* bare lands. While their proposal is carefully guarded and qualified, it tends to leave the impression that sufficient facts have been established to warrant so radical a departure as the elimination of protective forest cover. It is impossible to answer such a proposal in a few words, or even to enumerate more than the most poignant facts.

The paper lacks essential information on the geology and soils of the two areas, which would permit a student of such matters to rate the tendency toward surface run-off and erosion and the capacities of the water-sheds for deep ground storage by means of which the streams might be maintained through dry weather. It is not, for example, clearly stated that, in the case of the Wagon Wheel Gap water-sheds, the coarse crystalline character of the rock and the resultant porosity and thinness of the soil layer practically precluded the possibility of any surface run-off, and, at the same time, distinctly limited the capacity of the water-shed as a whole for storage in the ground, this so-called soil layer having an apparent average depth of only about 6 ft. In the case of the 3-in. rainfall of October, 1911, the impression is given that the peak of flow on both water-sheds was reached during, and by reason of, surface run-off, whereas this peak flow (although augmented by water falling in the streams) represented merely a saturated condition of the entire water-sheds and played only a small part in the entire flow which occurred subsequently. In such a case as this, and in fact in all the stream-flow phenomena of these water-sheds, it was clearly demonstrated that the surface conditions as modified by the existing forest could play but a very small part. This is far from being the case with mountain water-sheds in general.

It is obvious, of course, that the water-sheds at Wagon Wheel Gap do not represent quite the extreme of the porous, completely absorbing type of water-shed. The writer happens to be familiar with various sandhill regions. In Central Nebraska, for example—what with the porous sand as an absorbing mantle, the lack of dissection, the "pocket" conformation (which is characteristic of all sand dunes), and the great depth and storage capacity of the sand

reservoir—the streams that drain the region are extremely steady in their flow, and their annual "floods" probably do not exceed the annual low-water discharges by a ratio of more than about 3 to 1. The water-sheds at Wagon Wheel Gap had a high-low discharge ratio (daily, not peak discharges) of 12 to 1, and after deforestation Water-Shed *B* showed a ratio of 17 to 1. It is apparent, therefore, that the porous type of water-shed is not likely to show a ratio of more than 25 to 1, over a long period of years.

The omission by the authors of many pertinent facts with regard to the California areas discussed is decidedly confusing. Nothing is said as to the geology and soils of the two water-sheds. From a slight knowledge of Southern California foothill conditions and from the map presented (Fig. 2), the writer assumes that the lower portions of the two water-sheds have relatively deep but impermeable soils, the surface run-off of which contributes largely to the quick discharges after storms, while the upper portions are relatively rocky and barren. It is probably in the latter portion that most of the absorption of the voluminous winter rains occurs, by reason of which the streams (especially the Santa Anita with its second high area on Mt. Wilson) are maintained throughout most, but not all, of the summer months when no rain is recorded. Because of the fact that many of the winter rains in California are gentle and prolonged, there is, however, for the entire water-shed areas greater opportunity for absorption and storage than occurs with quick summer rains or, in many cases, with the melting of accumulated snow. Probably only in their lower reaches, therefore, do these water-sheds represent that completeness of surface run-off which is associated with arid climatic conditions and which is characteristic of the entire foothill fringe of the Western mountainous area and the adjacent plains and deserts.

There are much more extreme conditions than those cited, as in the case of the extremely impervious shaly or clay soils of the Bad Lands of Dakota and Montana. However, even without this extremely impervious type of soil, essentially all stream flow tends to be in the form of surface run-off and the streams "go dry" completely almost every summer and often throughout the greater part of each year. This is quite largely the result of incapacity to form humus and to enrich the surface soil therewith, which, in turn, can probably be ascribed to dry atmospheric conditions more than any other one factor. In short, all organic litter is oxidized so rapidly that there can be no accumulation, except possibly on a fraction of the area on which brush clumps mechanically hold litter and protect it from the sun. There is no chance of developing on such sites a forest canopy in the true sense, although scattered trees generally establish themselves in the less extreme conditions. In the intervening spaces between clumps of brush or trees, the bare mineral soil is generally exposed, these spaces comprising from 30 to 90% of the entire surface. Inevitably, the result is that surface run-off is never entirely precluded and that any effect from the destruction of the vegetation and existing partial humus mantle changes run-off conditions only in degree and not in kind.

The two types of water-shed drawn upon by the authors, therefore, represent almost complete antitheses, the one showing a high absorbing capacity

and a relatively steady flow and low ratio between flood discharge and summer minima, the other being in the "surface run-off" class, exhibiting little ability to utilize whatever deep storage the soil and geology of the areas provide, and frequently "going dry" in the summer, so that its high-low discharge ratio is in many years infinity, and on the average very high.

Between these two classes of water-sheds lies the greater proportion of the forested areas of the United States both in the Western mountains and in the more humid East. Although, of course, depending on the manner in which the surface cover has been treated, the high-low ratios for a large proportion of the water-sheds in the country run from 50:1 to 150:1. Under the more humid conditions which permit the actual development of a normal forest, and which, at the same time (by the more rapid processes of weathering) rarely permit extremely porous soils to exist, an entirely different situation exists. It seems undeniable, if one accepts human experience and observation as worth anything at all, that in one important particular the stream-flow situation is different from that in the two cases cited. This is in the matter of increased, rather than decreased, summer flow as a result of the long-time effects of forest cover and of humus upon the absorptive capacity of the soil.

If, for example, one can picture the difference between 3% run-off from a well-forested area, and 50 to 80% run-off from burned, pastured, and cultivated areas, during rains which pour from 1 in. to 3 in. of water upon the soil in as many hours, one obtains the basis for an entirely different conception of the value of forest cover. This value is obtained, necessarily, by deep storage of the additional water which gets into the ground through a good layer of humus. Much of this storage is undoubtedly beyond the reach of tree roots, so that the surface layer may become completely dried through root absorption, and still the deeper reservoir remains filled and supplies springs throughout long periods of dry weather. It is obvious that this effect on the low-water flow (even admitting a decrease in total annual run-off) arises solely through increased surface absorption and the existence of a potential large reservoir in the deep soil and rock layers. In recent years, and from a multiplicity of sources, the facts regarding the increased absorption due to forest humus have been established beyond a shadow of doubt. The work[12] of Dr. W. C. Lowdermilk, too, has presented an entirely new and much more sane conception of the function of humus in water absorption by soils, so that one no longer thinks of the humus layer as a reservoir in itself, but as supplying the channels through which water may penetrate an otherwise largely impermeable soil.

That all these facts have not been brought together under one comprehensive water-shed experiment, in which the year-long regimen of streams was studied, is unfortunate, but it should scarcely destroy sanity. In fact, the weight of observation on this point is so great that it certainly cannot be set aside because of conflicting observations under conditions of arid climates and limited storage possibilities. The conditions cited by the authors

[12] "Influence of Forest Litter on Run-Off, Percolation, and Erosion, *Journal of Forestry*, Vol. XXVIII, No. 4, April, 1930.

are in no sense "normal," although they may possibly be representative of considerable areas in the West.

The authors, while admitting the occurrence of severe erosion within a few months after the Fish Creek fire (which erosion filled the stream channel with débris and did "considerable damage to orchards, railroads, and highways" below the water-shed), tend to minimize its importance and state in their "Conclusions" that "so far as the surface features of the Fish Creek Basin were concerned the erosion did not destroy any present use." This bland statement of the harmless effect of one fire begs the question and is misleading. Erosion is a time-consuming process—it normally proceeds almost imperceptibly so far as "surface features" are concerned, and destructive erosion gains headway only after the "harmless" type of erosion has proceeded for a considerable time. Messrs. Hoyt and Troxell have shown that the beneficial effects of the fire denudation began to decrease after a couple of years. It would require repeated fires and almost complete elimination of all vegetative cover to maintain such benefits to stream flow. That destructive erosion would soon follow such treatment is a foregone conclusion, and there is only one logical outcome to the use of the methods proposed by the authors for obtaining greater stream flow—to remove the entire soil mantle of the water-sheds and with it every obstacle to quick surface run-off.

It seems scarcely probable that even the people of Southern California would place such stress on increased population and increased water supplies that they would choose to have their entire foothills region assume the barrenness of the upper slopes of Mt. Wilson. Nor is it conceivable that any engineer will argue, in view of the reservoir difficulties which are daily being brought to their attention, that surface run-off carrying a heavy burden of silt is equivalent in practical usefulness to a corresponding volume of clear, subsurface flow. There may be communities in dire need of additional water, but it is not believed there are any that can afford to obtain it by a method so wholly destructive in its design.

As far as the general proposition of "protection forests" on mountainous water-sheds is concerned, progressive erosion as the result of deforestation or complete denudation is the crux of the situation. It is rarely that areas will be found as immune to surface run-off and erosion as those at Wagon Wheel Gap, and it may be seriously questioned whether these areas would stand repeated burnings, and (as a logical sequence) grazing of the herbaceous vegetation which sprang up, without showing a great change in respect to these soil-destroying processes. At the other extreme, there is, chronically, some erosion despite the efforts of a feeble vegetative cover to maintain itself, and with the kinds of soils which usually prevail in the foothills areas, unimportant erosion may quickly develop into deep gullying. In areas of the intermediate class, there will be no less a tendency toward soil loss once the forest is removed, except as delayed by the greater depth of humidified soil and the richer vegetation which that soil and the more humid conditions will support. Once this "reserve" has been lost through a conscious effort to keep down the forest, the unprotected soil will give way in much the same manner.

It is for the conservation of the soil resource quite as much as the water resource or timber resource that the forester argues the need of "protection forests." In young mountainous regions, Nature has decreed that the one is indispensable to the other and that the building of soil in the first instance is dependent on the roots of trees and on lower forms of vegetation which grow on rock surfaces; then, on the chemical action of the organic matter which they deposit; and, finally, upon the protection from erosion which is furnished by the complete humus mantle. The forest, in turn, can develop as individual units and, finally, as a solid "stand" only as these soil-forming steps progress and as the storage capacity of the soil increases. Eventually, deep soil deposits are obtained which far surpass in storage capacity the requirements of the protecting forest.

Has the time arrived when Man may assume the audacity to start reversing these great constructive processes of Nature, for the sake of a temporary benefit, to water a few paltry additional acres of crops or to permit a few more people to congregate in the cities? The Engineering Profession is not lacking in audacity, and it frequently defaces and defies Nature for the sake of a temporary gain. When such a proposal as that of Messrs. Hoyt and Troxell is being seriously put forward, it is time for the profession to ask itself whether or not it is truly giving heed to the interests of humanity.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# WIND-BRACING IN STEEL BUILDINGS

## SECOND PROGRESS REPORT OF SUB-COMMITTEE NO. 31, COMMITTEE ON STEEL, OF THE STRUCTURAL DIVISION

### Discussion

BY MESSRS. O. G. JULIAN, AND JACOB FELD

O. G. JULIAN,[61] M. AM. SOC. C. E. (by letter).[61a]—The methods given by the Sub-Committee for the calculation of moments and deflections due to wind loads are ingenious and easy of application. The practicability of these methods should lead to their general use for the determination of moments and deflections resulting from gravity as well as from lateral loads.

If the component parts of a frame are so joined together that, when the frame is subjected to heavy lateral loads, the relative rotation of the members intersecting at any one joint is negligible, it appears obvious that the structure will act as a rigid frame with respect to the dead and live loads. The stresses due to "negative" moments in girders and columns resulting from gravity loads should be given more consideration than that required by the usual building codes. Conclusion (3) of the report has to do with "members or details subjected to wind stress only." If the "negative" moments resulting from gravity loads are given proper consideration, it would appear that all members and details subjected to wind loads are also subjected to gravity loads.

In some cases the stiffness added to girders by the floors may be negligible. However, in case the floor is concrete, the entire floor system may act as a unit with the girders, making the stiffness ratio, of the horizontal members to the columns, very high.[62] The evaluation of this stiffening effect

NOTE.—The report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, was presented at the meeting of the Structural Division, New York, N. Y., January 21, 1932, and published in February, 1932, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: May, 1932, by Messrs. L. J. Mensch, Robins Fleming, Rudolph P. Miller, C. M. Goodrich, Albert Smith, Hugh L. Dryden, L. E. Grinter, P. L. Pratley, Frederick Martin Weiss, and A. J. Hammond; and August, 1932, by Messrs. David Cushman Coyle, S. P. Wing, A. H. Finlay, J. D. Gedo, V. A. Vanoni and M. P. White, and H. V. Spurr.

[61] Engr. with Jackson & Moreland, Cons. Engrs., Boston, Mass.

[61a] Received by the Secretary June 23, 1932.

[62] See, also, "Modern Framed Structures," by Johnson, Bryan, and Turneaure, Pt. II, Tenth Edition, p. 563 *et seq.*

might be a profitable field for study. The writer does not understand that it is the Sub-Committee's intent to classify substantial concrete floors as partitions and, therefore, neglect their effect on moment calculations.

The advisability of allowing higher stresses in rivets and bolts than in other parts of the structure (see Conclusion (3)) is open to question. The most conservatively designed parts of the structure should be the connections, because they probably require the greatest "factor of ignorance." Calculations given elsewhere[63] indicate that the usual allowable working stresses in rivets subjected to shear may be somewhat too high.

The importance of Conclusion (7) should be emphasized. It appears clear that main members intersecting at comparatively stiffer joints will assume more than their share of moment, while those intersecting at relatively flexible joints will assume less than their share. The use of knuckle connections where rigidity is required appears to be questionable practice, and should be discouraged.

The "reasonable stress" in tension rivets, which is stated in Conclusion (9) to be "entirely satisfactory" may be taken as seven-tenths of the yield-point strength of the rod from which the rivet is manufactured,[64] or about 26 000 lb. per sq. in. This is approximately equal to the intensity of the initial tension incident to driving.

Step 1 in the Appendix should state: "The column shears and moments are, respectively, proportional to the $\frac{I}{L^3}$ and $\frac{I}{L^2}$ of the columns."

Step 4 is really a repetition of Step 1, the unbalance in shear being used as a hypothetical lateral load to make $H = 0$. The correcting column shears and moments resulting from this hypothetical lateral load are, therefore, respectively proportional to the $\frac{I}{L^3}$ and $\frac{I}{L^2}$ of the columns.

The saving in labor resulting from estimating the final moments in the columns by a semi-graphical method is not apparent. The writer finds it just as easy and somewhat surer to continue the calculations indicated by Fig. 3 for several more cycles.

JACOB FELD,[65] ASSOC. M. AM. SOC. C. E. (by letter).[65a]—The report of the Sub-Committee is a great step toward the development of rational wind-bracing design methods. With the exception of two items (Recommendations (3) and (4)), there seems to be no possible criticism of, or dissension from, the report. The method of determining lateral deformation is especially useful and deserving of praise.

Recommendation (3)—that members or details subjected to wind stress only should be designed for no greater stress than is allowed for dead load or for dead load and live load—will not result in economy or consistency of

[63] *Proceedings*, Am. Soc. C. E., April, 1932, p. 677 *et seq.*

[64] *Bulletin No. 210*, Experiment Station, Univ. of Illinois, and *Proceedings*, Am. Soc. C. E., April, 1932, p. 677 *et seq.*

[65] Cons. Engr., New York, N. Y.

[65a] Received by the Secretary June 27, 1932.

design. The writer cannot see why members carrying loads other than wind loads should be allowed greater unit combined stresses than wind-bracing members carrying no other loads.

To test Recommendation (4)—that the Cross method as described in the Appendix might be used as a rapid and accurate method of moment determination—the writer applied such method to several designs of building frames from 20 to 26 stories in height, executed in his office during the past few years. Apparently, for the ordinary tier building (designed for live loads from 40 to 120 lb. per sq. ft.) the ratio of $\frac{I}{L}$ of girders to that of columns is far less than that in the example shown in Fig. 3. The convergence of the series of approximation is not very rapid. Where long spans of girders framed into heavy columns, no definite convergence could be seen after six sets of steps, as outlined in the recommended method.

For bents of uniform story height, the writer has used the following method, the results from which are close to those obtained by the Cross method as recommended in this report, and require less time for computation:

1.—Design columns and girders for dead load and live load.

2.—Assign a relative value to each column, equal to its moment of inertia divided by the cube of its free length. The free length is taken as the story height, less the depth of the girder. The relative values change slightly where column sections change; but for a 20-story frame, average ratios at the fifth and fifteenth stories are sufficient, each applied to ten stories.

3.—Distribute the total wind shears carried by a bent in each story to all the columns in the bent in direct ratio to the relative values.

4.—Assume points of inflections in the columns at the mid-point of each free length and compute the column moments for design purposes.

5.—Compute the girder moments from the equilibrium at successive joints and with the assumption that the points of inflection are at mid-span.

Where unequal stories are encountered, the Cross method, or a modified form of the Wilson-Maney method (using results obtained by the foregoing steps for the regular stories, except the two directly above the non-regular story), is suggested. Where column sections are revised for wind stresses combined with dead load and live load, the results must be adjusted. For structures of 25 stories and less, such adjustment is seldom required.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# DESIGN OF LARGE PIPE LINES

## Discussion

BY ROBERT E. GLOVER, ESQ.

ROBERT E. GLOVER,[52] ESQ. (by letter).[52a]—A rigid connection between stiffener ring and pipe shell will enforce equality of displacements along their common surface of contact. Mr. Schorer has shown how the enforcement of equality of radial displacements results in relieving the circumferential stresses in the pipe shell in the vicinity of the stiffener ring at the expense of increased tension in the ring and local bending stresses in the pipe shell.

It is of interest also to consider the results that follow from the enforcement of equality of circumferential displacements. In this case the bending stresses in the ring are relieved at the expense of circumferential stresses induced locally in the pipe shell, and the net effect is much the same as if a certain part of the pipe shell acted as a flange of the stiffener ring.

In order to ascertain approximately how much of the pipe shell might be considered to act with the stiffener ring to resist bending, the writer calculated, by the theory of elasticity,[53] the strains produced in a flat plate of indefinite extent subjected to forces in its plane such that the intensity, measured in units of force per unit of length, varied as the ordinate of a sine curve. These strains were compared with those calculated for a plate of the same thickness and of finite width under the usual assumption of uniform stress distribution, and the following formula was obtained:

$$\beta = \frac{\lambda}{\pi\left[2 - \frac{(1-m)^2}{2}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (209)$$

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna; January, 1932, by Messrs. Paul Bauman and L. J. Larson; February, 1932, by Messrs. C. M. Orr and Edward J. Bednarski; April, 1932, by Messrs. H. C. Boardman, R. L. Templin and R. G. Sturm, and F. Knapp; and May, 1932, by Messrs. C. P. Vetter, L. E. Grinter, and Oren Reed.

[52] Engr., U. S. Bureau of Reclamation, Denver, Colo.

[52a] Received by the Secretary May 24, 1932.

[53] The stress function used for this purpose may be found in Paragraph 238 of "Applied Elasticity," by J. Prescott.

in which, $\beta$ represents the width of a uniformly stressed plate that is equivalent to the infinite plate in its ability to resist deformation, and $\lambda$ represents the wave length of the applied forces.

In order to apply this result to the present problem it was necessary to represent the moment diagram shown on the author's Fig. 8 as nearly as possible by a single sinusoidal variation, and a curve was chosen for the purpose such that its ordinates were proportional to cosine ($3u$). If it is recalled that forces are transmitted from web to flange in a beam by means of shear stresses the intensity of which is proportional to the rate of change of bending moment with respect to length, it may be seen that for the idealized curve the circumferential forces between stiffener ring and pipe shell will be proportional to sine ($3u$). Under these conditions:

$$\lambda = \frac{2\pi r}{3} \qquad (210)$$

and,

$$\beta = \frac{2r}{3\left[2 - \frac{(1-m)^2}{2}\right]} \qquad (211)$$

If Poisson's ratio, $m$, is given the value, 0.28, this relation reduces to,

$$\beta = 0.383\, r \qquad (212)$$

The effect of circumferential equalization on the bending stresses in the stiffener ring may be shown by the following computation based on the data of the author's example, thus: $\beta = 0.383\, r = 0.383 \times 60 = 22.98$ in.; the area of the section is $1 \times 12 + 22.98 \times 0.25 = 17.75$ sq. in.; the center of gravity of the section is 4.27 in. from the inner surface of the pipe shell; $R = 64.27$ in.; the moment of inertia is 293.5 in.$^4$; and the section modulus $= \frac{293.5}{12.25 - 4.27} = 36.75$ in.$^3$

Hence, $M = 0.010\, Q\, R = 0.010 \times 317\,000 \times 64.27 = 203\,700$ lb-in., and the bending stress in the outer fiber of the stiffener ring is, $f_1 = \frac{203\,700}{36.75} = 5\,540 \frac{\text{lb.}}{\text{in.}^2}$. A comparison of these figures with those of the author shows a smaller value of $R$, a smaller moment, and a reduced bending stress as results accruing from the equalization of the circumferential displacements.

## APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

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**The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.**

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**BAHMEIER, HERMAN FREDERICK,** Pinedale, Wyo. (Age 43). Refers to E. B. Debler, J. R. Iakisch, E. Mead, R. L. Parshall, W. W. Weir.

**BARANOWSKY, PAUL,** Ozone Park, N. Y. (Age 21). Refers to H. P. Hammond, E. J. Squire.

**BAYLISS, LAWRENCE ALFRED,** Hawthorne, Cal. (Age 26). Refers to I. L. Collier, J. W. Miller, C. C. More, R. G. Tyler.

**BECHTEL, GEORGE MITCHELL,** Liberty, Kans. (Age 23). Refers to G. H. Hackmaster, R. E. Lawrence, W. C. McNown.

**BERNSTROM, CARL GUSTAV,** Rensselaer, N. Y. (Age 20). Refers to E. R. Cary, L. W. Clark.

**BICKFORD, ANDREW ARTHUR,** Guatemala City, Guatemala. (Age 32). Chf. Engr. and Director of Water Supplies. Refers to A. M. Brosius, A. S. Hadley, S. F. Shaw, S. Vilar y Boy, L. F. Whitbeck.

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**BUSALACCHI, JOSEPH MICHAEL,** Milwaukee, Wis. (Age 23). Refers to C. U. Smith, F. W. Ullius, C. S. Whitney.

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**DeLaMATER, RAY SPENCER,** Wichita, Kans. (Age 21). Refers to L. E. Conrad, F. F. Frazier, M. W. Furr.

**EAGLE, HENRY CARLSON,** West Yellowstone, Mont. (Age 24). Refers to E. L. Grant, N. C. Grover, E. W. Schoder, F. J. Seery.

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**FINNIE, ALEXANDER GORDON,** Fishers Island, N. Y. (Age 31). Gen. Supt., Fishers Island Corporation. Refers to A. F. Brooke, W. N. Brown, O. B. French, J. R. Lapham, M. T. Singleton, H. F. Wiedeman.

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**HALL, WILLIAM GOLDSBOROUGH,** East Orange, N. J. (Age 21). Refers to L. W. Clark, T. R. Lawson.

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**JAMES, ARTHUR WILLIAM,** Lincoln Park, Mich. (Age 24). Refers to W. J. Henderson, S. C. Hollister.

**JENNY, ARTHUR BERNHARDT,** Dumont, N. J. (Age 22). Refers to H. H. Hatch, J. I. Leonard, C. T. Schwarze, J. B. Whipple, J. Zimmer.

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**LEVAN, STEPHEN,** Bremen, Ind. (Age 30). Plant Inspector, Indiana State Highway Comm. Refers to G. E. Lommel, R. B. Wiley.

**LOUGHRAN, EUGENE VINCENT,** Astoria, N. Y. (Age 24). Refers to J. J. Costa, N. Oppenheim.

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**STONE, CHARLES ALFRED,** Boulder, Colo. (Age 22). Refers to E. O. Bergman, R. L. Downing, C. L. Eckel, E. W. Raeder, W. H. Thoman.

**STRAIGHT, ARTHUR LEROY,** Saylesville, R. I. (Age 23). Refers to C. D. Billmyer, J. L. Murray.

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**TROXLER, PAUL DEXTER,** Serres, Greece. (Age 27). Div. Engr., John Monks & Sons –Ulen & Co. Refers to J. A. Anderson, M. A. Butler, A. B. Christensen, R. W. Gausmann, R. H. Keays, M. R. Keefe, E. W. Wilson.

**VOLINO, MICHAEL ANTHONY,** New York City. (Age 22). Refers to H. R. Codwise, H. P. Hammond, E. J. Squire.

**VREELAND, ROBERT PAUL, Jr.,** Maplewood, N. J. (Age 21). Refers to C. T. Bishop, T. Crane, J. C. Tracy.

**WARNER, FAYETTE SAMUEL,** Rochester, N. Y. (Age 38). Valuation Engr. with Edward J. Cheney, Engr., New York City. Refers to C. Benham, Jr., L. W. Clark, C. C. Cooman, N. H. Davidson, P. M. Sax, P. F. Stephens, P. H. Wilson, R. M. Yager.

**WAROFF, LOUIS,** New York City. (Age 21). Refers to E. G. Hooper, C. T. Schwarze.

**WARREN, PAUL EDWARD,** Boulder, Colo. (Age 21). Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder.

**WHALEN, LOUIS ALOYSIUS, Jr.,** Rochester, N. Y. (Age 23). Refers to L. W. Clark, H. O. Sharp.

**WILLARD, ROGER HERSPBARGER,** Burkittsville, Md. (Age 25). Refers to J. H. Gregory, J. T. Thompson.

**WINGATE, JOHN METCALF,** Paulsboro, N. J. (Age 23). Refers to H. K. Preston, R. W. Thoroughgood.

**WOOD, HAROLD AARON,** Gallup, N. Mex. (Age 28). Refers to J. L. Burkholder, J. H. Dorroh, R. G. Hosea, H. C. Neuffer, A. W. Newcomer, W. B. Ream.

**WYLIE, MARSHAL JESSE,** Gallup, N. Mex. (Age 25). Refers to J. L. Burkholder, J. H. Dorroh, R. G. Hosea, H. C. Neuffer, A. W. Newcomer, W. B. Ream.

## FOR TRANSFER

### FROM THE GRADE OF JUNIOR

**FOX, FREDERICK CARL, Jr.,** Jun., Poughkeepsie, N. Y. (Elected June 26, 1931). (Age 27). Asst. Civ. Engr., New York State Dept. of Public Works, Div. of Highways. Refers to J. Barnett, F. W. Burnett, A. de H. Hoadley, L. G. Holleran, L. S. Hulburd, F. P. McKibben, R. T. Retz, H. A. Schauffler, W. C. Taylor.

**HAINOVSKY, NICHOLAS JOHN,** Jun., Havana, Cuba. (Elected July 16, 1928). (Age 32). Refers to D. J. Cloward, B. A. Etcheverry, S. T. Harding, G. D. Munch, I. M. Nelidov.

**INGRAHAM, RICHARD WILLIAM,** Jun., Napa, Cal. (Elected Nov. 10, 1930). (Age 32). Asst. Inspector of Constr., Mare Island, (Cal.) Navy Yard. Refers to R. J. Buck, C. H. Cotter, E. D. Graffin, J. F. Greathead, T. C. Hatton, G. E. Sievers, W. C. West, R. Whitman, G. R: Young.

**NEWMAN, HARRY,** Jun., Brooklyn, N. Y. (Elected Oct. 15, 1923). (Age 31). Asst. Sec. Engr. with Board of Transportation, New York City. Refers to B. A. Hodgdon, H. Holbrook, W. A. Mellny, J. H. Myers, N. Oppenheim, J. A. Ruddy.

**NOYES, HAYDON THOM,** Jun., New Rochelle, N. Y. (Elected Aug. 17, 1931). (Age 29). With Turner Constr. Co., New York City. Refers to E. K. Abberley, R. W. Boyd, H. C. Dinney, R. E. Goodwin, F. O. X. McLoughlin, A. C. Nichols, H. C. Paddock.

**PLOCK, HENRY JOHN,** Jun., Mineola, N. Y. (Elected June 7, 1926). (Age 32). Instructor, Coll. of City of New York. Refers to H. F. Butler, R. E. Goodwin, F. O. X. McLoughlin, J. R. Smith, L. W. Weed.

**SAWYER, WILLIAM LINCOLN,** Jun., Gainesville, Fla. (Elected Oct. 1, 1928). (Age 27). Instructor in Civ. Eng., Univ. of Florida. Refers to C. C. Brown, W. C. Huntington, T. M. Lowe, P. L. Reed, A. H. Ross, T. C. Shedd, C. C. Wiley.

**TSAI, FANG-YIN,** Jun., Peiping, China. (Elected Oct. 10, 1927). (Age 31). Prof. of Civ. Eng., National Tsing Hua Univ. Refers to N. A. Richards, T. F. Russum, C. M. Spofford, H. V. Spurr, C. H. Sutherland.

**WOODS, EDWIN MARECHAL,** Jun., Philadelphia, Pa. (Elected June 4, 1928). (Age 31). Refers to M. Brooks, J. Cantley, A. F. Holler, G. V. S. Keely, M. Y. Neely.

*The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.*

# AMERICAN SOCIETY OF CIVIL ENGINEERS

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# AMERICAN SOCIETY OF CIVIL ENGINEERS

## COMING MEETINGS

### BOARD OF DIRECTION MEETINGS

**October 3-4, 1932:**
A Quarterly Meeting will be held at Atlantic City, N. J.

### FALL MEETING

### October 5, 6, 7, and 8, 1932

### ATLANTIC CITY, N. J.

**October 5, 1932:**
**Morning.**—Technical Meeting.
**Afternoon.**—Technical Meeting.
**Evening.**—Dinner and Entertainment.

**October 6, 1932:**
**Morning.**—Technical Division Sessions.
**Afternoon.**—Luncheon and Bridge for Ladies; Golf, Fishing, Boating, etc., for Members.
**Evening.**—Reception, Entertainment, and Dance.

**October 7, 1932:**
**Morning.**—Technical Division Sessions.
**Afternoon.**—Golf, Fishing, Boating, etc.

**October 8, 1932:**
**Morning.**—Trip to Philadelphia, Pa.; Luncheon and Inspection of Pennsylvania Railroad Improvements and Other Points of Interest.
**Afternoon.**—Bus Excursion to Valley Forge.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.